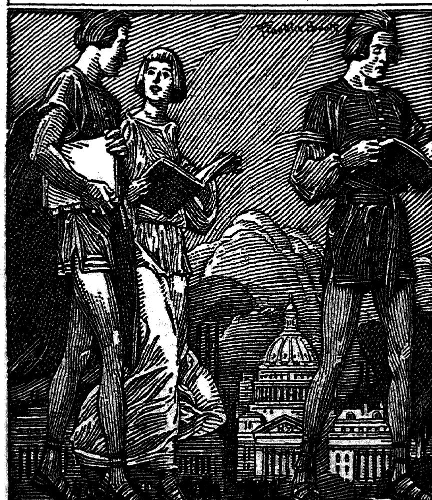

MODERN TIMBER DESIGN

BY

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IN THE REPRINTING OF THIS BOOK, THE RECOMMENDATIONS OF THE WAR PRODUCTION BOARD HAVE BEEN OBSERVED FOR THE CONSERVATION OF PAPER AND OTHER IMPORTANT WAR MATERIALS. THE CONTENT REMAINS COMPLETE AND UNABRIDGED.

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PREFACE

Part of the material in this volume was used by the author for a course in Modern Timber Design given as one of the Engineering, Science, and Management defense training courses at Tulane University. Since then, the original material has been revised, new subjects have been covered, and many illustrative problems have been added. The present volume is used in a comprehensive course in Timber Design given to first semester seniors at the Agricultural and Mechanical College of Texas. Since a course in Indeterminate Structures is offered in the last semester of the senior year, this book does not include the design of this type of wood structure. However, all the basic information needed by the designing engineer is included. The book is intended not only as a textbook but also as a guide and reference for practicing engineers.

Because wood is different from other materials and because it is nonhomogeneous, most of the design data have been derived from test results. It has been the function of the personnel at the Forest Products Laboratory at Madison, Wisconsin, which is a part of the United States Department of Agriculture, to develop the necessary data for scientific design in wood, and most of the available information is a result of their tests and interpretations. Naturally, then, no book on timber design can be written without due acknowledgment to the staff of the Forest Products Laboratory. Generous use has been made of their publications.

Grateful acknowledgment is also given to the National Lumber Manufacturers' Association, Timber Engineering Company, West Coast Lumbermen's Association, Southern Pine Association, Douglas Fir Plywood Association, and the Service Bureau of the American Wood Preservers' Association.

The author is grateful for the criticisms and suggestions offered by Professor C. E. Sandstedt in his review of the contents.

HOWARD J. HANSEN

COLLEGE STATION, TEXAS
January, 1943.

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CHAPTER I

CHARACTERISTICS AND PROPERTIES OF WOOD

1. General. There are more than 180 species of trees grown in the United States that may be considered commercially important. However, the number suitable for structural purposes is relatively small. Nineteen distinct groups have been assigned working stresses; these are ash, beech, birch, cedar, chestnut, cypress, elm, fir, gum, hemlock, hickory, larch, maple, oak, pecan, pine, redwood, spruce, and tupelo. Many of these comprise several species that have individual commercial names and are distinguishable because of their locality of growth and their physical and mechanical properties. Of all the groups, two are used most extensively for structural lumber. These are the southern pines, which grow in the Atlantic and Gulf States and as far north as West Virginia and Kentucky; and the Douglas firs, which grow in the coastal region of Washington, Oregon, and California and in the inland region of Montana, Idaho, Washington, and Oregon.

In using any species in the design and construction of timber structures, it is not necessary for the engineer to know a great deal about the chemical composition of wood. However, an understanding of this subject will often aid him in selecting the proper species under various conditions of use. Of more importance to the designing engineer are the mechanical properties of the different species, the factors affecting their strength, the intelligent use of their assigned working stresses, and the characteristics which make their use in design different from other structural materials.

2. Hardwoods and Softwoods. All species are divided into two general classes, namely, hardwoods and softwoods. However, no definite degree of hardness divides the two groups. Differences in structure, appearance, size, and quality keep the two groups separated. Many of the hardwoods such as oak and hickory are very hard whereas many of the softwoods are soft; but there are many exceptions, such as basswood, which is classed as a hardwood but is among the softest of native woods. Longleaf pine, on the other hand, is classed as a softwood but is about as hard as any hardwood. Softwoods are also called "conifers" because most species are cone bearing. The two groups are

most accurately differentiated by calling all trees with needle leaves softwoods and all broad-leaved trees hardwoods.

3. Structure. Wood is composed of elongated cells whose framework is cellulose. The cells are cemented together by lignin, and their arrangement in the tree greatly affects the appearance and properties of the different species. The cross section of most trees will show certain features that are common to all trees; Figure 1 illustrates the important parts that make up the structure of any tree.

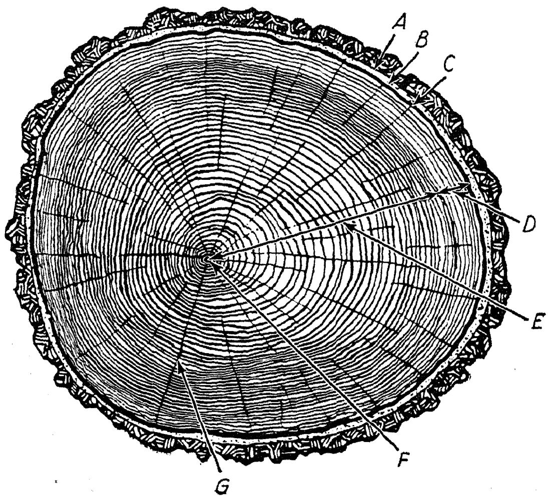


FIG. 1. Typical cross section of a tree.

A is dry, dead tissue called outer bark which serves as a protective coating.

B is the moist, soft inner bark which carries food from the leaves to other parts of the tree.

C is a microscopic layer just inside the inner bark which is called the cambium. It is here that new wood and bark cells are formed.

D is the sapwood, which is light in color. Its function is to carry sap from the roots of the tree to the leaves.

E is the heartwood, which is usually dark in color. It is formed by a gradual change in sapwood and is inactive in the tree.

F is the pith. It is here that the new wood growth for twigs takes place.

G represents wood rays which connect the various parts of a tree for the storage and movement of food.

4. Heartwood and Sapwood. Heartwood is formed by a gradual change in sapwood. When wood is used under conditions conducive to

decay a large amount of heartwood in the cross section of a piece is desirable because the heartwood of all species is more durable than the sapwood. However, if the material is to be treated, sapwood is preferable because it absorbs preservatives more readily.

The heartwood of some species is considerably more durable than others. Table 1 may be used as a guide in comparing a number of species.

5. Growth Rings. As shown in Figure 1, concentric rings start at the center or pith of the tree and continue outward toward the bark. Each ring represents the growth of the tree during one year. This growth is in the cambium, so that the new wood is added just inside this layer and tends to push the bark outward. When trees grow in a variable climate, it is possible to distinguish one growth ring from another because the cells formed during a cold season are different from those formed during a warm season.

6. Springwood and Summerwood. Each annual ring is divided into two layers. The inner one, called springwood, is developed during the first part of the growing season. It is composed of large cells with thin walls and is usually lighter in color than the summerwood. The outer layer, called summerwood, consists of smaller cells with thicker walls and is the dark portion of the annual ring. It is heavier and stronger than springwood and has an important effect upon the strength properties of most species.

7. Density. Density is a definite criterion of the strength of softwoods. It is determined by the rate of tree growth and the amount of summerwood present. This means that the strength of a piece of wood is being measured by the amount and distribution of wood substance, which is the material making up the cell walls. The specific gravity of this wood substance is the same for all woods, 1.54, and all woods would be of the same specific gravity throughout were it not for the difference in the arrangement and size of cells and the thickness of the cell walls.

In grading softwood lumber for structural purposes, the number of rings per inch radially and the proportion of summerwood in the cross section of the piece are considered as part of the specifications. Material having a specified minimum number of rings per inch is termed "close-grained" and material that has in addition one-third or more of summerwood is termed "dense."

8. Grain. The wood from trees of rapid growth will have wide annual growth rings and is called coarse-grained. On the other hand wood from slow-growing trees has narrow growth rings and is often called close-grained. Straight grain and cross grain are used to describe wood in which the fibers are parallel to, or at an angle to, the sides of the piece.

Table 1. Durability of Heartwood of Various Species

[Wood Handbook, U. S. Dept. Agr., 1935]

Heartwood durable even when used under conditions that favor decay.	Cedar, Alaska. Cedar, eastern red. Cedar, northern white. Cedar, Port Orford. Cedar, southern white. Cedar, western red. Chestnut. Cypress, southern. Locust, black. Osage-orange. Redwood. Walnut, black. Yew, Pacific.
Heartwood of intermediate durability but nearly as durable as some of the species named in the high-durability group.	Douglas fir (dense). Honey locust. Oak, white. Pine, southern yellow (dense).
Heartwood of intermediate durability.	Douglas fir (unselected). Gum, red. Larch, western. Pine, southern yellow (unselected). Tamarack.
Heartwood between the intermediate and the nondurable group.	Ash, commercial white. Beech. Birch, sweet. Birch, yellow. Hemlock, eastern. Hemlock, western. Hickory. Maple, sugar. Oak, red. Spruce, black. Spruce, Engelmann. Spruce, red. Spruce, Sitka. Spruce, white.
Heartwood low in durability when used under conditions that favor decay.	Aspen. Basswood. Cottonwood. Fir, commercial white. Willow, black.

The slope of grain, expressed as a ratio between a one-inch deviation of the grain from the side of the piece and the distance within which this deviation occurs, is taken into consideration in the specifications for structural timbers because it has a marked effect on the strength of the piece.

Grain, however, usually refers to the appearance of the piece. Lumber sawed in such a manner that the annual rings form an angle of 45° or

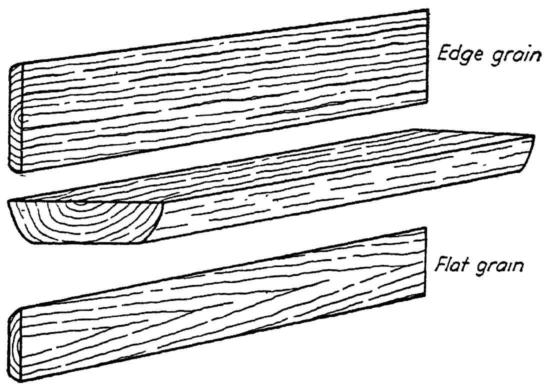


FIG. 2. Flat and edge grain lumber.

more with the surface of the piece is called edge grain, vertical grain, or rift-sawn in softwoods and quarter-sawn in hardwoods. Flat grain or plain sawn refers to lumber that has been sawed approximately tangent to the growth rings; i.e., the rings form an angle of less than 45° with the surface of the piece.

9. Knots. There are many types and classifications of knots, depending upon the appearance of the knot on a sawed surface and whether the knot is the result of a limb that was alive or dead when the tree was cut. The engineer should be concerned with the influence of the knot on the strength of a piece of wood rather than with the type of knot. Knots effect the strength because it is necessary for the grain to deviate from its regular direction in passing around them and because checking often results in and around them as the piece loses moisture. The influence on strength is determined by the location of the knot and the area that it occupies in the cross section of a piece.

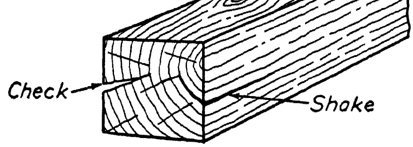


FIG. 3. Knots, checks, and shakes.

The rules for grading structural lumber limit the size of knots, depending upon the working stress assigned to the individual grade and whether or not the piece is to be used as a beam or column.

10. Checks and Shakes. A check is a lengthwise separation of the wood across the annual growth rings, and a shake is a separation along the grain between the annual growth rings. A number of checks and shakes in a piece will reduce its working value in shear. The amount and location of checks and shakes are limited in structural timber grades. In some grading rules the unit working stress in horizontal shear is the same for all structural grades, but higher or lower horizontal shear stresses may be secured within certain limits by specifying the working stress that is desired.

11. Moisture Content. Wood contains a considerable amount of free water within its cell walls. After a tree is cut it begins to lose moisture, and the moisture content continues to drop throughout the entire manufacturing process. This moisture content is defined as the

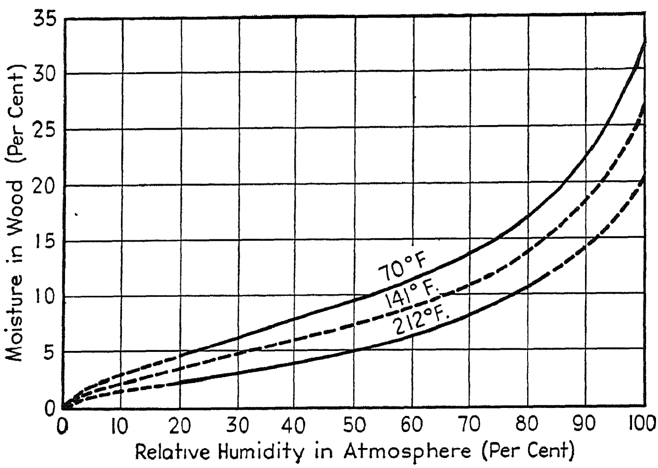


CHART 1. Humidity-moisture content relationship at three temperatures. (*Wood Handbook, U. S. Dept. Agr., 1935.*)

weight of water contained in wood expressed as a percentage of the weight of the oven-dry wood. When put into use, wood will continue to dry until it is in equilibrium with the surrounding atmosphere. By equilibrium moisture content is meant the ultimate moisture content that wood will attain, depending upon the temperature and the relative humidity in the atmosphere. The accompanying chart shows the average relationship between the moisture content of wood and the relative humidity of the surrounding atmosphere at three temperatures.

Wood is said to have reached the fiber saturation point when all the free water in the cells is evaporated and the cell walls are still saturated. This point is generally reached when the moisture content is between 24 and 30 per cent. As wood dries beyond the fiber saturation point it increases in strength. However, such properties as toughness or shock resistance decrease because wood in a dry condition will not bend as much as wood in a green condition. In commercial structural grades the increase in strength due to drying is to a large extent offset by the influence of defects that develop during seasoning.

12. Shrinkage. Shrinkage takes place after the fiber saturation point has been reached. Accompanied by a reduction in moisture content is a reduction in the size of a piece, for as wood dries it also shrinks. Wood shrinks most tangentially or in the direction of the annual growth rings

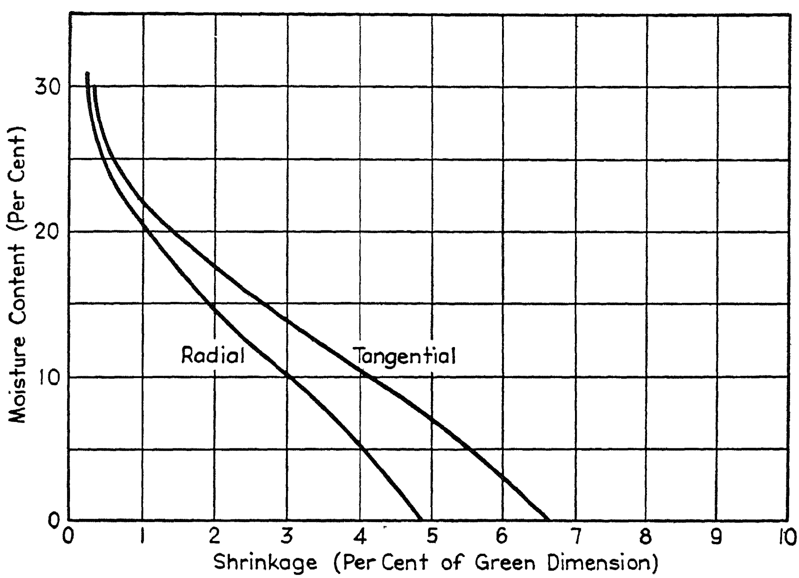


CHART 2. Average moisture-shrinkage curves for southern pine and Douglas fir. (*Wood Handbook, U. S. Dept. Agr., 1935.*)

and about one-half to two-thirds as much radially or across the annual rings. The longitudinal shrinkage in most woods is negligible. Tests at the Forest Products Laboratory have established average shrinkage values for all species. Chart 2, which shows typical moisture-shrinkage curves for Douglas fir and southern pine, may be used for estimating the amount of change in dimension that will take place with a change in moisture content.

Naturally, the engineer should attempt to secure lumber that is seasoned to its ultimate moisture content. However, it is seldom possible to obtain large timbers fully seasoned and, therefore, a certain amount of shrinkage is to be expected and provided for in the design. Methods of design and construction that provide vertical bearings across the grain should be avoided. For instance, the use of joists placed directly on top of a girder increases the vertical height of wood used and results in increased shrinkage. If the joists bear on ledger strips fastened to the sides of a girder, there is a reduction in vertical height and less shrinkage. In the construction of mill buildings and similar structures this same idea can be carried out to eliminate part of the shrinkage by using metal post caps to separate the upper column from the lower column. This means that the girder is not being used as a bearing for the upper column. The use of a cast iron pintle, bearing on a metal post cap, will produce the same effect and will also allow the girder to bear directly on the lower column.

Structural joints fabricated with green wood and loaded before seasoning takes place should be inspected at regular intervals while the timber is undergoing a reduction in moisture content. If nails become loose, simply resetting the old nails is not sufficient; the joint should be reenforced with additional nails. The design details for bolted joints and for joints employing connectors, given in a later chapter, provide reductions in the safe design loads for different conditions of seasoning, but these joints should be inspected for serious checks that might weaken them, and for loose bolts.

13. Sizes. A table of sizes for the various items of lumber manufactured is included in the appendix. For material 2, 2½, 3, and 4 in. in nominal thickness, the actual thickness or dressed size becomes 1⅝, 2⅛, 2⅝ and 3⅝ in. respectively; the actual width is ⅜ in. off the nominal width through 7 in. and ½ in. off the nominal width above that dimension. For material 5 in. thick and thicker by 5 in. wide and wider, the actual thickness and width are ½ in. off the nominal. Calculations for areas and moments of inertia are based on actual sizes. However, lumber is sold on the basis of the contents of the nominal size expressed in terms of board feet. A board foot is the contents of a volume $12 \times 12 \times 1$ in.

14. Mechanical Properties. Several hundred thousand tests have been performed at the Forest Products Laboratory on small clear specimens from 164 species. The results of these tests and discussions relating to the effects on strength induced by such variables as locality of growth, position in tree, rate of growth, knots, cross grain, pitch pockets, moisture content, duration of stress, and kiln drying appear in

Table 2. Strength Properties of Various Species

[Compiled from Tech. Bull. 479, U. S. Dept. Agr., 1935]

Name	Modulus of Rupture (lb. per sq. in.)		Modulus of Elasticity (1000 lb. per sq. in.)		Maximum Crushing Strength Parallel to Grain (lb. per sq. in.)		Compression Perpendicular to Grain Prop. Limit (lb. per sq. in.)		Maximum Shearing Strength Parallel to Grain (lb. per sq. in.)	
	Green	Dry [1]	Green	Dry	Green	Dry	Green	Dry	Green	Dry
Hardwoods										
Ash, white	9,600	15,400	1,460	1,770	3,990	7,410	810	1,410	1,380	1,950
Beech	8,600	14,900	1,380	1,720	3,550	7,300	670	1,250	1,290	2,010
Chestnut	5,600	8,600	930	1,230	2,470	5,320	380	760	800	1,080
Elm, rock	9,500	14,800	1,190	1,540	3,780	7,050	750	1,520	1,270	1,920
Gum, black	7,000	9,600	1,030	1,200	3,040	5,520	600	1,150	1,100	1,340
Hickory, pignut	11,700	20,100	1,650	2,260	4,810	9,190	1,140	2,450	1,370	2,150
Maple, sugar	9,400	15,800	1,550	1,830	4,020	7,830	800	1,810	1,460	2,330
Oak, red	8,300	14,300	1,350	1,820	3,440	6,760	760	1,250	1,210	1,780
Oak, white	8,300	15,200	1,250	1,780	3,560	7,440	830	1,320	1,250	2,000
Softwoods										
Cedar, western red	5,100	7,700	920	1,120	2,750	5,020	340	610	710	860
Cypress, southern	6,600	10,600	1,180	1,440	3,580	6,360	500	900	810	1,000
Douglas fir, coast type	7,600	11,700	1,550	1,920	3,890	7,420	510	910	930	1,140
Douglas fir, Rocky Mountain type	6,400	9,600	1,180	1,400	3,000	6,060	450	820	880	1,070
Hemlock, eastern	6,400	8,900	1,070	1,200	3,080	5,410	440	800	850	1,060
Hemlock, western	6,100	10,100	1,220	1,490	2,990	6,210	390	680	810	1,170
Larch, western	7,500	11,900	1,350	1,710	3,800	7,490	560	1,080	920	1,360
Pine, longleaf	8,700	14,700	1,600	1,990	4,300	8,440	590	1,190	1,040	1,500
Pine, shortleaf	7,300	12,800	1,390	1,760	3,430	7,070	440	1,000	850	1,310
Redwood, second growth	6,100	8,300	1,000	1,120	3,280	5,240	350	640	730	930
Spruce, Sitka	5,700	10,200	1,230	1,570	2,670	5,610	340	710	760	1,150
Spruce, white	5,600	9,800	1,070	1,340	2,570	5,470	290	570	690	1,080

[1] Material at 12 per cent moisture content.

Technical Bulletin 479, published by the United States Department of Agriculture. Table 2 lists the results of these tests on some of the more important woods.

This table is to be used only as a guide in comparing the mechanical properties of different species. The values shown therein, together with the results of supplementary investigations on the effect of defects, are the basis for assigning working stresses to structural grades of lumber.

CHAPTER II

WORKING STRESSES

15. Unit Working Stresses for Structural Lumber. Working stresses for structural lumber have been set forth in the grading rules published by associations of lumber manufacturers whose products are of a species usable for structural purposes. Table 3 includes stress grades that are standard commercial grades for a number of species. The unit stresses should be used in design only after they have been modified in accordance with the accompanying text.[1]

16. Conditions of Exposure. Of first consideration in the use of the above working stresses is the condition of exposure under which the lumber is to be used. The full working stress values may be used under severe conditions of exposure if the lumber has adequate heartwood or if it has been given an approved preservative treatment. If the lumber is lacking in heartwood or is untreated, the amount by which the working stresses should be reduced to adapt them to exposure conditions is usually left to the judgment of the engineer. The amount of reduction may depend upon the extent to which the exposure favors decay, the required life of the structure, the frequency and thoroughness of inspection, and the initial and replacement costs. Table 4 is a guide for reducing working stresses of structural grades used under different conditions of exposure.

17. Duration of Load. When loads remain on the structure for a short time only, the working stresses, except modulus of elasticity, may be increased somewhat. An example of this condition is found in designing for wind loads in addition to dead and live loads. In this case the working stresses may be increased 50 per cent providing the resulting structural members are not smaller than those designed for dead and live loads alone.

18. Impact. The working stresses may be used without regard to impact, unless the impact stress exceeds the allowable live-load stress.

19. Tension. For members in direct tension, allowable stresses for the respective grades are the same as for extreme fiber stress in bending.

20. Compression Parallel to the Grain. Values in compression parallel to the grain apply only to posts, struts, or columns whose un-

[1] See Appendix A.

Table 3. Working Stresses for Important Structural Woods

Association and Effective Date of Grading Rules	Species	Grade	Allowable Unit Stresses (lb. per sq. in.)				Modulus of Elasticity
			Extreme Fiber in Bending	Horizontal Shear	Compression Perpendicular to Grain	Compression Parallel to Grain	
Southern Cypress Mfgrs. Assn. 1/1/41	Tidewater red cypress	1400#f tidewater red cypress	1,400	120	300		1,200,000
		1100#f tidewater red cypress	1,100	100	300		1,200,000
		1200#c tidewater red cypress				1,200	
		1000#c tidewater red cypress				1,000	
National Hardwood Lumber Assn. 1/1/41	Southern cypress	1400#f southern cypress	1,400	120	300		1,200,000
		1100#f southern cypress	1,100	100	300		1,200,000
		1200#c southern cypress				1,200	
		1000#c southern cypress				1,000	
National Hardwood Lumber Assn. 1/1/41	Rock elm	1800#f rock elm	1,800	120	500		1,300,000
		1600#f rock elm	1,600	120	500		1,300,000
		1400#f rock elm	1,400	120	500		1,300,000
		1200#f rock elm	1,200	100	500		1,300,000
		1300#c rock elm				1,300	
		1200#c rock elm				1,200	
		1000#c rock elm				1,000	
West Coast Lumbermen's Assn. 1/1/41	Douglas fir (coast region)	Dense select structural	1,800	120	380	1,300	1,600,000
		Select structural	1,600	100	345	1,200	1,600,000
		1200#f framing and joist	1,200	100	325		1,600,000
		900#f framing and joist	900	100	325		1,600,000
		No. 1 timbers				1,100	
		No. 1 dimension				880	
Western Pine Assn. 4/1/41	Douglas fir (inland empire)	Select structural	1,800	120	380	1,466	1,600,000
		Structural	1,600	85	335	1,175	1,500,000
		Common structural	1,200	80	315	1,050	1,500,000

Northern Hemlock & Hardwood Mfgrs. Assn. 6/27/41	Eastern hemlock	Select structural	1,100	70	300	700	1,110,000
		1000 SG eastern hemlock	1,000	52	300		1,110,000
		900 SG eastern hemlock	900	52	300		1,110,000
		800 SG eastern hemlock	800	52	300		1,110,000
Southern Pine Inspection Bureau of Southern Pine Assn. 5/28/40	Longleaf southern pine	Select structural	2,000	100	380	1,450	1,600,000
		Prime structural	1,800	100	380	1,300	1,600,000
		Merchantable structural	1,600	100	380	1,200	1,600,000
		Structural square edge and sound	1,600	100	380	1,200	1,600,000
		No. 1 structural	1,400	100	380	1,000	1,600,000
		No. 1 L.L. dimension	1,400	100	380		1,600,000
		No. 2 L.L.—1050#f dimension	1,050	100	380		1,600,000
Southern Pine Inspection Bureau of Southern Pine Assn. 5/28/40	Shortleaf southern pine	Dense select structural	2,000	100	380	1,450	1,600,000
		Dense structural	1,800	100	380	1,300	1,600,000
		Dense structural square edge and sound	1,600	100	380	1,200	1,600,000
		Dense No. 1 structural	1,400	100	380	1,000	1,600,000
		No. 1 dense dimension	1,400	100	380		1,600,000
		No. 1 dimension	1,200	100	380		1,600,000
		No. 2 dense—1050#f dimension	1,050	100	380		1,600,000
		No. 2 medium grain—900#f dimension	900	100	380		1,600,000
California Redwood Assn. 5/1/40	Redwood	1600#f close-grained	1,600	80	267		1,200,000
		Dense select all-heart	1,400	80	267		1,200,000
		Select all-heart	1,200	70	267		1,200,000
		1200#c close-grained				1,200	
		1100#c close-grained				1,100	
		1000#c close-grained				1,000	
Northeastern Lumber Mfgrs. Assn. 4/1/38	Eastern spruce	1200#f eastern spruce	1,200	90	250		1,200,000
		1100#f eastern spruce	1,100	80	250		1,200,000
		1000#f eastern spruce	1,000	80	250		1,200,000
National Hardwood Lumber Assn. 1/1/41	Tupelo	1400#f tupelo	1,400	100	300		1,200,000
		1200#f tupelo	1,200	100	300		1,200,000
		1000#f tupelo	1,000	100	300		1,200,000
		900#c tupelo				900	

Table 4. **Per Cent Reduction in Working Stresses for Different Conditions of Exposure**

Kind of Stress	Decay Hazard None	Moderate	Severe
Extreme fiber in bending	100	85	71
Compression across grain	100	70	58
Compression parallel to grain	100	92	78
Horizontal shear	100	100	100
Modulus of elasticity	100	100	100

supported length does not exceed eleven times the least dimension of the cross section. For cases where the L/d ratio is greater than eleven, suitable column formulas must be used.

21. Compression Perpendicular to the Grain. The values for compression perpendicular to the grain apply to bearings 6 in. or more in length, located anywhere in the length of the piece, and to bearings of any length at the ends of beams or other members. If bearings are shorter than 6 in. and located 3 in. or more from the end of a timber, the stresses may be increased in accordance with the factors in Table 5.

Table 5. Increase in Working Stress for Different Lengths of Bearing

Length of Bearing (in.)	*Factor*	*Length of Bearing (in.)*	*Factor*
$\frac{1}{2}$	1.85	3	1.15
1	1.60	4	1.10
$1\frac{1}{2}$	1.45	6 or more	1.00
2	1.30		

For stress under a washer the same factor may be taken as for a bearing whose length equals the diameter of the washer.

22. Loads at an Angle to the Grain. When the direction of pressure is at an angle to the direction of the fibers in a piece of wood, the allowable unit stress must be calculated. The most recent formula developed for determining this stress is known as the Hankinson formula, and is as follows:

$$N = \frac{pq}{p \sin^2 \theta + q \cos^2 \theta}$$

N = allowable unit stress on the inclined surface.
p = unit stress in compression parallel to grain.
q = unit stress in compression perpendicular to grain.
θ = angle between the direction of the load and the direction of the grain.

CHAPTER III

FASTENINGS

23. General. Under load in wood joints, nails, spikes, screws, and bolts are stressed not only in shear but also in bending. This condition creates a non-uniform bearing pressure, and the determination of the stress distribution is an extremely difficult mathematical problem. The true solution has not been found theoretically; consequently, safe working loads have been established by a large number of tests.

The Forest Products Laboratory has developed, through hundreds of tests, formulas for calculating the safe loads with respect to the withdrawal and lateral resistance of nails, spikes and screws. The following is a presentation of these formulas.

NAILS

24. Withdrawal Resistance. The safe working load of a nail when the load is applied in the direction of its length is directly related to the specific gravity of the wood and the diameter of the nail. The general formula for the load required to withdraw common wire nails driven perpendicular to the grain in seasoned wood is:

$$P = 6900G^{5/2}D$$

P = ultimate load, in pounds per lineal inch of penetration.
G = specific gravity, based on oven-dry weight and volume when oven-dry.
D = diameter of nail, in inches.

A factor of safety of 6 being introduced, the safe working load becomes:

$$P_1 = 1150G^{5/2}D$$

Certain species will give somewhat higher values than those found by using the general equation above, and others will fall below the equation value. For longleaf and shortleaf pine the Forest Products Laboratory recommends using 80 per cent of the equation value for sixpenny and twelvepenny nails, 70 per cent for the twentypenny and larger sizes, and 75 per cent for sixteenpenny nails. For Sitka spruce the

values given by the formula should be increased 10 per cent, and for northern white pine an increase of 20 per cent is recommended.

When nails are driven into green wood and seasoning then takes place, their holding power is generally reduced, but insufficient test data make it impossible to establish a percentage reduction for the various species. In an important structural joint that has been loaded before seasoning takes place, simply resetting the old nails is not sufficient; additional nails should be added to reenforce the joint.

The withdrawal resistance decreases as the angle between the grain and direction of driving decreases. In the softer woods the holding power of nails driven into the end of the piece drops to 75 or 50 per cent of the value obtained from the above equation.

There are many types and sizes of nails. Table 6 lists those commonly used.

Table 6. Nail Sizes

[American Steel & Wire Co.]

Gage	1	2	3	4	5	6	7	8	9	10	$10\frac{1}{4}$
Diameter	.2830	.2625	.2437	.2253	.2070	.1920	.1770	.1620	.1483	.1350	.1314
Gage	$10\frac{1}{2}$	11	$11\frac{1}{2}$	12	$12\frac{1}{2}$	13	14	$14\frac{1}{2}$	15	$15\frac{1}{2}$	16
Diameter	.1278	.1205	.1130	.1055	.0985	.0915	.0800	.0760	.0720	.0673	.0625

Name	Size	2d	3d	4d	5d	6d	7d	8d	9d	10d	12d	16d	20d	30d	40d	50d	60d
	Length	1″	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3	$3\frac{1}{4}$	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	$5\frac{1}{2}$	6
Common	Gage	15	14	$12\frac{1}{2}$	$12\frac{1}{2}$	$11\frac{1}{2}$	$11\frac{1}{2}$	$10\frac{1}{4}$	$10\frac{1}{4}$	9	9	8	6	5	4	3	2
	No /lb.*	876	568	316	271	181	161	106	96	69	63	49	31	24	18	15	11
Siding	Gage	$15\frac{1}{2}$	$14\frac{1}{2}$	14	14	$12\frac{1}{2}$	$12\frac{1}{2}$	$11\frac{1}{2}$	$11\frac{1}{2}$	$10\frac{1}{2}$							
	No./lb.	1010	635	473	406	236	210	145	132	94							
Finishing	Gage	$16\frac{1}{2}$	$15\frac{1}{2}$	15	15	13	13	$12\frac{1}{2}$	$12\frac{1}{2}$	$11\frac{1}{2}$	$11\frac{1}{2}$	11	10				
	No./lb.	1351	807	584	500	309	238	189	172	121	113	90	62				
Flooring	Gage	...	...	...	...	11	11	10	10	9	8	7	6				
	No /lb.	...	...	...	...	157	139	99	90	69	54	43	31				
Fence	Gage	...	...	...	10	10	9	9	8	7	6	5					
	No /lb.	...	...	...	142	124	92	82	62	50	40	30					

Std. Barbed	Size & Length	$\frac{3}{4}$″	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2
Roofing	Gage	13	12	12	12	11	11	10	10	9
Nails	No./lb.*	714	469	411	365	251	230	176	151	103

Round Spikes	Size	10d	12d	16d	20d	30d	40d	50d	60d	7″	8	9	10	12
Flathead	Length	3″	$3\frac{1}{4}$	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	$5\frac{1}{2}$	6	7	8	9	10	12
Diamond Pt.	Gage	6	6	5	4	3	2	1	1	$\frac{5}{16}$″	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
	No./lb.*	41	38	30	23	17	13	10	9	7	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$

* Approximate

25. Lateral Resistance. The general equation for the safe lateral load driven into the side grain of seasoned lumber is $P = KD^{3/2}$.

P = safe lateral load, in pounds per nail.
K = a constant varying with species.
D = diameter of nail, in inches.

Table 7. Specific Gravity of Important Structural Woods *

Hardwoods Species	Specific Gravity	Softwoods Species	Specific Gravity
Ash, white	0.64	Cedar, western red	0.34
Chestnut	.45	Cypress, southern	.48
Elm, rock	.66	Douglas fir	.51
Gum, black	.55	Hemlock, eastern	.43
Gum, red	.53	Hemlock, western	.44
Hickory	.72	Larch, western	.59
Maple, hard	.68	Pine, southern	.58
Maple, soft	.51	Redwood	.39
Oak	.68	Spruce	.39

* Based on volume and weight when oven-dry.

The equations recommended for various species (Table 8) are based on a factor of safety of 1.6 for proportional-limit loads and about 6 for maximum loads.

The equations are based on a penetration, into the wood member holding the point, of two-thirds the length of the nail. For a metal-to-timber connection the safe lateral resistance determined from the equations given in the tabulation may be increased 25 per cent. When the nail is driven into the end of a piece, 60 per cent of the equation values should be used. If nails are driven into unseasoned wood, the equation values should be reduced by 25 per cent.

SPIKES

26. Withdrawal and Lateral Resistance. Common wire spikes are nothing more than large nails, except that for corresponding lengths spikes have larger diameters. The same formulas as given for nails can be used in calculating the safe withdrawal and lateral resistances of spikes. In determining the withdrawal resistance, however, two-thirds of the length of the point should be neglected.

27. Illustrative Problems. (*a*) Find the withdrawal resistance per lineal inch of penetration for a 12d nail and a spike $\frac{3}{8}$ in. in diameter when driven perpendicular to the grain in seasoned southern pine.

$$P = 1150G^{2\frac{1}{2}}D$$

Nail: $P = 1150 \times 0.58^{2\frac{1}{2}} \times 0.1483 \times 0.80 = 34.9$ lb.

Spike: $P = 1150 \times 0.58^{2\frac{1}{2}} \times 0.375 \times 0.70 = 77.3$ lb.

(*b*) Find the number and size of spikes to carry a lateral load of 3000 lb. between a 3-in. and 8-in. piece of seasoned southern pine.

Table 8. Equation for Computing Safe Lateral Resistance for Common Wire Nails Driven Perpendicular to the Grain of Wood at 15 Per Cent Moisture Content Expressed in Pounds per Nail

[Wood Handbook, U. S. Dept. Agr., 1935]

Species of Wood	Equation
Softwoods:	
Cedar, northern and southern white Fir, balsam and commercial white Hemlock, eastern Pine, lodgepole, ponderosa, sugar, northern white, and western white Spruce, Engelmann, red, Sitka, and white	$P = 900D^{3/2}$
Cedar, Alaska, incense, Port Orford, and western red Cedar, eastern red Cypress, southern Douglas fir (Rocky Mountain region) Hemlock, western Pine, Norway Redwood Tamarack	$P = 1125D^{3/2}$
Douglas fir (coast region) Larch, western Pine, southern yellow	$P = 1375D^{3/2}$
Hardwoods:	
Aspen and largetooth aspen Basswood Butternut Chestnut Cottonwood, black and eastern Poplar, yellow	$P = 900D^{3/2}$
Alder, red Ash, black Birch, paper Elm, American and slippery Gum, black, red, and tupelo Hackberry Magnolia, cucumber Magnolia, evergreen Maple, bigleaf Maple (soft), red and silver Sugarberry Sycamore	$P = 1250D^{3/2}$
Ash, commercial white Ash, Oregon Beech Birch, sweet and yellow Cherry, black Elm, rock	$P = 1700D^{3/2}$
Hickory, true and pecan Honey locust Locust, black Maple (hard), black and sugar Oak, commercial red and white Walnut, black	$P = 1700D^{3/2}$

The required penetration of the spike is two-thirds of its length. Select a spike 8 in. long and check this length for the required penetration, which is $\frac{2}{3} \times 8$ or 5.33. This result plus the thickness of the 3-in. piece is 7.95 in., and the 8-in. spike will satisfy. A spike 8 in. long will be $\frac{3}{8}$ in. in diameter and the safe load for one spike is determined from the formula $P = 1375D^{3/2}$.

$$P = 1375 \times (0.375)^{3/2} = 316 \text{ lb.}$$

$$\text{No. required} = \frac{3000}{316} = 10$$

No definite rules have been set up for the spacing of nails or spikes, but their spacing is largely dependent upon the resistance to splitting of the wood used. A general precaution to avoid splitting is to use a nail or spike with a diameter of not more than one-seventh the thickness of the lumber. Based on this rule, the maximum diameter of spike in the above problem should be $\frac{1}{7} \times 2.625 = 0.375$ in.

COMMON SCREWS

28. Withdrawal Resistance. As with nails, the withdrawal resistance of screws is related to the density of the piece and the diameter of the screw. Tests made by the Bureau of Standards have made it possible to express this relationship as follows:

$$P = 10{,}200G^2D$$

where P = ultimate load, in pounds per inch of total length,
G = specific gravity, based on oven-dry weight and volume when tested,
D = diameter of screw, in inches.

Introducing a factor of safety of 6, the safe load becomes

$$P_1 = 1700G^2D$$

The equation is based on screws inserted into the side grain of seasoned material and with a depth of penetration of not less than two-thirds the length of the screw. When screws are driven parallel to the grain or into the end of a piece, use 75 per cent of the value obtained from the above equation. The equation values apply to the following screw lengths and gages.

Table 9. Screw Lengths and Gages Applicable in Formula for Withdrawal Resistance

Screw Length (in.)	*Gage Limits*	*Screw Length (in.)*	*Gage Limits*
$\frac{1}{2}$	1– 6	2	7–16
$\frac{3}{4}$	2–11	$2\frac{1}{2}$	9–18
1	3–12	3	12–20
$1\frac{1}{2}$	5–14		

Table 10. Equations for Computing the Safe Lateral Loads for Wood Screws in Wood at 15 Per Cent Moisture Content Expressed in Pounds per Screw

[Wood Handbook, U. S. Dept. Agr., 1935]

Species of Wood	Equation
Softwoods:	
Cedar, northern and southern white Fir, balsam and commercial white Hemlock, eastern Pine, lodgepole, ponderosa, sugar, northern white, and western white Spruce, Engelmann, red, Sitka, and white	$P = 2100D^2$
Cedar, Alaska, incense, Port Orford, and western red Cedar, eastern red Cypress, southern Douglas fir (Rocky Mountain region) Hemlock, western Pine, Norway Redwood Tamarack	$P = 2700D^2$
Douglas fir (coast region) Larch, western Pine, southern yellow	$P = 3300D^2$
Hardwoods:	
Aspen and largetooth aspen Basswood Butternut Chestnut Cottonwood, black and eastern Poplar, yellow	$P = 2100D^2$
Alder, red Ash, black Birch, paper Elm, American and slippery Gum, black, red, and tupelo Hackberry Magnolia, cucumber Magnolia, evergreen Maple, bigleaf Maple (soft), red and silver Sugarberry Sycamore	$P = 2900D^2$
Ash, commercial white Ash, Oregon Beech Birch, sweet and yellow Cherry, black Elm, rock Hickory, true and pecan Honey locust Locust, black	$P = 4000D^2$
Maple (hard), black and sugar Oak, commercial red and white Walnut, black	$P = 4000D^2$

29. Lateral Resistance. The safe lateral load can be expressed by the following equation:

$$P = KD^2$$

P = safe lateral load, in pounds per screw.
K = a constant, varying with species.
D = diameter of shank, in inches.

The equations for various species are given in Table 10.

The equations are based on a penetration into the member receiving the point of seven times the screw diameter. For a metal-to-timber connection increase the safe loads 25 per cent.

LAG SCREWS

30. General. Lag screws are large screws with square heads requiring a wrench to turn them into the wood. They should be inserted into pre-bored holes from 40 to 85 per cent of the shank diameter, depending upon the species used. For Douglas fir and southern pine, the percentage is between 60 and 75.

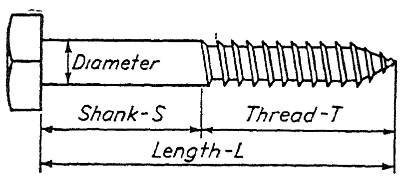


FIG. 4. Lag screw.

The following recommendations are based on tests conducted at the Forest Products Laboratory. The data given apply only to lag screws with a yield point of 45,000 lb. per sq. in. and to their use in seasoned lumber. For joints used under conditions where they will be alternately wet and dry use three-fourths of the load and if damp or wet most of the time use two-thirds.

31. Withdrawal Resistance. The formula for expressing the ultimate withdrawal load combines the influence of the specific gravity and the diameter of the screw and introduces a constant for different species so that

$$P = KG^{3/2}D^{3/4}$$

P = ultimate withdrawal load per inch of penetration of the threaded portion of the lag screw in the side grain of seasoned wood.
K = a constant, becoming 7500 for Douglas fir, southern pine, redwood, white pine, and white oak.
D = diameter of shank, in inches.
G = specific gravity of wood, based on oven-dry weight and volume when oven-dry.

Table 11. Lag Screw Sizes

Diameter (in.) D *	Root Diameter (in.) R *	Threads per Inch (*)	Thickness of Cut Washer (in.) W	Diameter of Pilot Hole (in.) For Threaded Portion	For Shank
$\frac{1}{4}$	0.173	10	0.0625	$\frac{5}{32}$	Same
$\frac{5}{16}$	.227	9	.0625	$\frac{3}{16}$	as
$\frac{3}{8}$	.265	7	.0781	$\frac{1}{4}$	Shank
$\frac{7}{16}$	.328	7	.0781	$\frac{5}{16}$	Diameter.
$\frac{1}{2}$	.371	6	.1093	$\frac{3}{8}$	(D)
$\frac{5}{8}$	.471	5	.1406	$\frac{7}{16}$	
$\frac{3}{4}$	.579	$4\frac{1}{2}$	.1562	$\frac{9}{16}$	
$\frac{7}{8}$	.683	4	.1718	$\frac{5}{8}$	
1	.780	$3\frac{1}{2}$	.1718	$\frac{3}{4}$	

Length of Threaded Portion (T) *(in.)* *

Diameter (in.) D	Length (L)—measured from under head to end of lag screw (in.) 2 & $2\frac{1}{4}$	$2\frac{1}{2}$ & $2\frac{3}{4}$	3 & $3\frac{1}{2}$	4 & $4\frac{1}{2}$	5 & $5\frac{1}{2}$	6 & $6\frac{1}{2}$	7 & $7\frac{1}{2}$	8 & $8\frac{1}{2}$	9 & $9\frac{1}{2}$	10	11	12	13	14	15	16
$\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	$5\frac{1}{4}$						
$\frac{5}{16}$	"	$1\frac{5}{8}$	"	"	"	"	"	"	"	"						
$\frac{3}{8}$	"	"	"	"	"	"	"	"	"	"	$5\frac{1}{2}$	6				
$\frac{7}{16}$	"	$1\frac{3}{4}$	"	"	"	"	"	"	"	"	"	"				
$\frac{1}{2}$	"	"	"	"	"	"	"	"	"	"	"	"	$6\frac{1}{2}$	7	$7\frac{1}{2}$	8
$\frac{5}{8}$	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
$\frac{3}{4}$			"	"	"	"	"	"	"	"	"	"	"	"	"	"
$\frac{7}{8}$			"	"	"	"	"	"	"	"	"	"	"	"	"	"
1			"	"	"	"	"	"	"	"	"	"	"	"	"	"

* Data from Federal Specification FF-B-561.

Introducing a factor of safety of 5, the safe load becomes

$$P = 1500G^{3/2}D^{3/4}$$

If the lag screw is inserted into the end of a piece use three-fourths of the value obtained from the above equation.

32. Lateral Resistance. The lateral resistance parallel to the grain for a lag screw driven into the side grain of seasoned lumber is

$$P = KD^2$$

P = safe load per screw.
K = a constant varying with species.
D = diameter of shank of screw, in inches.

Table 12. Equations for Computing Safe Lateral Resistance of Lag Screws

[Tech. Bull. 597, U. S. Dept. Agr.]

Group	Species of Wood	Equation	Group	Species of Wood	Equation
1	Cedar, northern and southern white Fir, balsam and commercial white Hemlock, eastern Pine, ponderosa, sugar, northern white, and western white Spruce, Engelmann, red, Sitka, and white	$P_1 = 1500D^2$	3	Ash, black Birch, paper Douglas fir (coast type) Elm (soft), American and (grey) slippery Gum, black, red, and tupelo Larch, western Maple (soft), red and silver Pine, southern yellow Sycamore	$P_1 = 1900D^2$
2	Aspen, and largetooth aspen Basswood Cedar, Alaska, Port Orford, and western red Chestnut Cottonwood, black and eastern Cypress, southern Douglas fir (Rocky Mountain type) Hemlock, western Pine, Norway Redwood Tamarack Yellow poplar	$P_1 = 1700D^2$	4	Ash, commercial white Beech Birch, sweet and yellow Elm, rock Hickory, true and pecan Maple (hard), black and sugar Oak, commercial red and white	$P_1 = 2200D^2$

The equations for safe lateral resistance are based on a ratio of cleat thickness to shank diameter of 3.5 and a penetration of the shank to the plane of contact between the two members. For other conditions see Charts 3 and 4.

The equations also assume that the depth of penetration into the main member will be nine times the diameter of the screw. If the penetration is less, the working load should be determined by multiplying the equation by the ratio of the actual penetration to that required. No increase in safe load is allowed if the penetration is greater than nine times the diameter.

If metal cleats are used the equation values can be increased by 25 per cent.

If the force on a lag screw driven into the side grain is applied perpendicular to the grain of the wood, the safe load may be computed by multiplying the safe load parallel to the grain by the ratio of the strength perpendicular to the grain to that parallel to the grain for bolts having an L/D ratio of 12. This procedure will be covered in a later article on bolts. If the screw is driven into the end grain use two-thirds of the value obtained for side grain.

The spacings and distances may be taken as the same as those given for bolts.

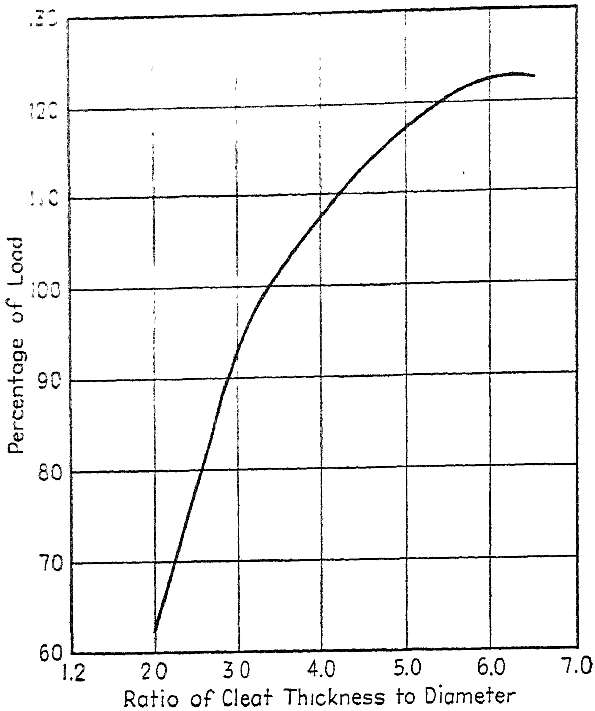


CHART 3. Influence on load of the ratio of cleat thickness to the diameter of the shank of a lag screw.

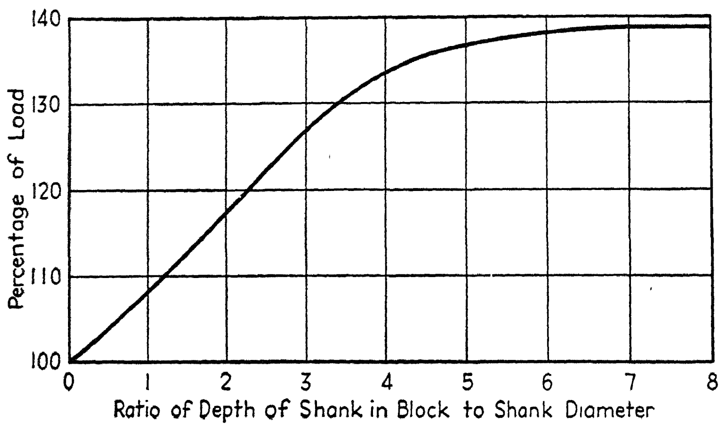


CHART 4. Influence on load of the ratio of the length of shank in block to the shank diameter.

DRIFT BOLTS

33. General. A drift bolt is a round or square steel or iron pin without threads. Drift bolts are usually driven into holes about ⅛ in. smaller than their diameter. The only test results available are those with respect to withdrawal resistance, and these have not been extensive enough to suggest safe working loads.

34. Withdrawal Resistance. The resistance to withdrawal of round drift bolts driven perpendicular to the grain into seasoned wood may be expressed by the formula

$$P = 6000G^2D$$

P = maximum withdrawal load per lineal inch of penetration.
G = specific gravity, based on weight and volume when oven-dry.
D = diameter of drift bolt, in inches.

Some species will fall above the equation value and others below it. For southern pine 80 per cent of the equation value should be used; and a factor of safety of 6 being introduced, the safe working load may be expressed as $P_1 = 800G^2D$.

BOLTS

35. Action of a Bolt. The stress distribution in the wood of a bolted joint is greatest at the edges of the timber, as shown in Figure 5. At

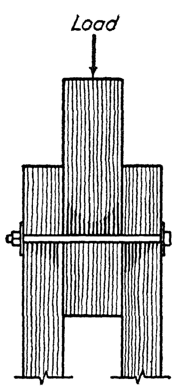


FIG. 5. Distribution of stress in a bolted joint.

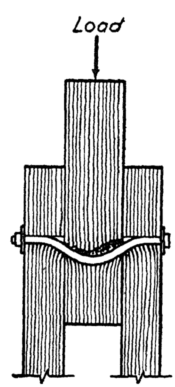


FIG. 6. Bolted joint at ultimate load.

ultimate load the deformation of a slender bolt in a timber joint is illustrated in Figure 6, which brings out the fact that because bending is

present the actual bearing is distributed over a rather small part of the length of the bolt.

Tests at the Forest Products Laboratory revealed the existence of a definite relationship between the length and the diameter of the bolt and the proportional limit of the average bearing value of the wood. This means that the average bearing value of the wood may be expressed in terms of the ratio of the length of the bolt to its diameter in the main member of the joint. Furthermore, the tests showed that this relationship exists regardless of the size of the bolt; i.e., a ½-in. bolt in a timber 4 in. thick will have the same average bearing value as a 1-in. bolt in a timber 8 in. thick. This is true for loads parallel to the grain; however, when the load is applied perpendicular to the grain, it is necessary, within certain limits, to apply a corrective factor depending upon the size of the bolt.

36. Basic Stresses. From the tests performed at the Forest Products Laboratory, basic stresses have been developed for calculating the safe loads for bolted joints. These stresses are for use in the design of bolted

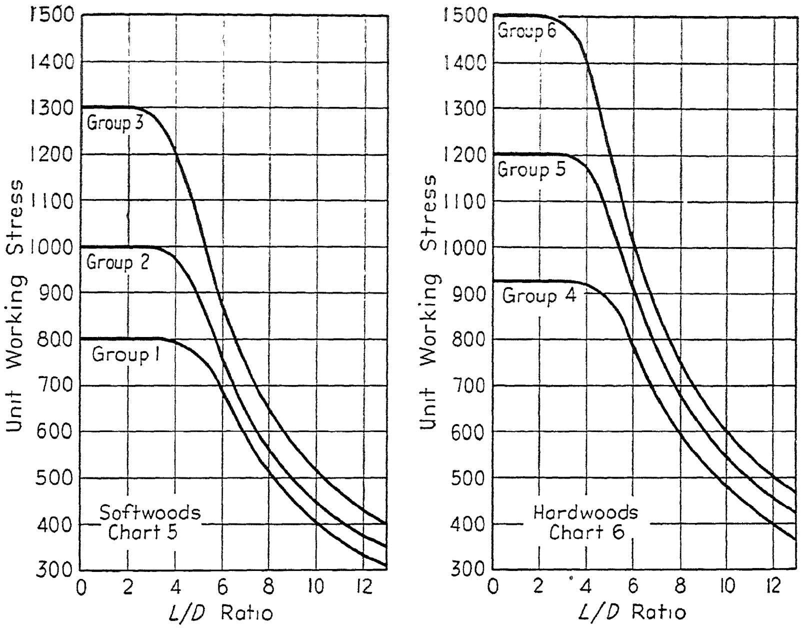
Table 13. Basic Stresses for Calculating Safe Loads for Bolted Joints

Group	Species of Wood	Basic Stress (lb. per sq. in.) *Parallel with the Grain*	Basic Stress (lb. per sq. in.) *Perpendicular to the Grain*
Softwoods (conifers):			
1	Cedar, northern and southern; fir, balsam and commercial white; hemlock, eastern; pine, lodgepole, ponderosa, sugar, northern white, and western white; spruce, Engelmann, red, Sitka, and white	800	150
2	Cedar, Alaska, incense, Port Orford, and western red; Douglas fir (Rocky Mountain); hemlock, western; pine, Norway	1000	200
3	Cedar, eastern red; cypress, southern; Douglas fir (coast region); larch, western; pine, southern yellow; redwood; tamarack	1300	275
Hardwoods (broad-leaved species):			
4	Ash, black; aspen and largetooth aspen; basswood; birch, paper; butternut; chestnut; cottonwood, black and eastern; poplar, yellow	925	175
5	Alder, red; elm, American and slippery; gum, black, red, and tupelo; hackberry; magnolia, cucumber and evergreen; maple, bigleaf, and red and silver (soft); sugarberry; sycamore	1200	250
6	Ash, commercial white and Oregon; beech; birch, sweet and yellow; cherry, black; elm, rock; hickory, true and pecan; honey locust; locust, black; maple (hard), black and sugar; oak, commercial red and white; walnut, black	1500	400

joints only and are not to be confused with the working stresses for structural lumber.

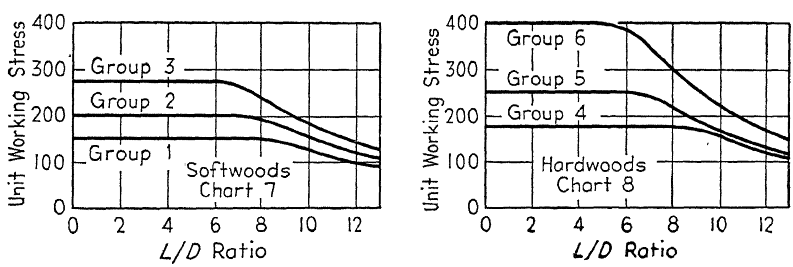
The basic stresses apply irrespective of the grade of lumber used and are for seasoned timbers used in a dry inside location.

37. Varying *L/D* Ratio. The following charts show the working stress to use in calculating the safe load of a bolted joint for different



CHARTS 5 AND 6. Safe working stresses for bolted joints for any L/D ratio.

Load applied parallel to grain of main member. Values based on use of metal splice plates. For wood splice plates use 80 per cent of these values. Common bolts—yield point 45,000 lb. per sq. in.



CHARTS 7 AND 8. Safe working stresses for bolted joints for any L/D ratio.

Load applied perpendicular to grain of main member. Values are based on use of metal or wood splice plates and common bolts. Yield point—45,000 lb. per sq. in.

L/D ratios. The charts are based on a three member joint (double shear) and the length of the bolt is measured as the thickness of the main member. When wood splice plates are used they are to be one-half the thickness of the main member.

Table 14 lists the diameter factors of different bolts to be used as a multiplier in determining the safe load when the load is applied perpendicular to the grain.

Table 14. Diameter Factors for Loads Perpendicular to the Grain.

Diameter of Bolt (in.)	*Diameter Factor*	*Diameter of Bolt (in.)*	*Diameter Factor*
$\frac{1}{4}$	2.50	$1\frac{1}{4}$	1.19
$\frac{3}{8}$	1.95	$1\frac{1}{2}$	1.14
$\frac{1}{2}$	1.68	$1\frac{3}{4}$	1.10
$\frac{5}{8}$	1.52	2	1.07
$\frac{3}{4}$	1.41	$2\frac{1}{2}$	1.03
$\frac{7}{8}$	1.33	3 and over	1
1	1.27		

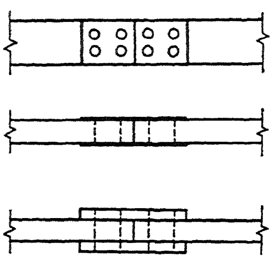


FIG. 7. Bolted joint with load parallel to grain.

38. Illustrative Problems. (*a*) Consider a tension joint as shown in Figure 7, with metal and wood splice plates. The main member is 6 in. thick ($5\frac{1}{2}$ in. actual) and consists of two pieces of seasoned southern pine joined by these splice plates and eight common bolts $\frac{7}{8}$ in. in diameter. The joint is to be used in a dry inside location.

The L/D ratio is $\dfrac{5.5}{0.875} = 6.28$ and for this ratio Chart 5 gives a safe stress with metal splice plates of 825 lb. per sq. in. With wood splice plates the safe stress is $825 \times 0.8 = 660$ lb. per sq. in.

Safe load for one bolt $= 825 \times 5.5 \times 0.875 = 3970$ lb.
Safe load for four bolts $= 3970 \times 4 = 15{,}880$ lb.

With wood splice plates each one-half the thickness of the main member the

Safe load for one bolt $= 3970 \times 0.8 = 3175$ lb.
Safe load for four bolts $= 3175 \times 4 = 12{,}700$ lb.

(*b*) The joint shown in Figure 8 consists of a main member 6 in. thick with splice plates 3 in. thick and is seasoned southern pine to be used in a dry inside location. Four common bolts $\frac{7}{8}$ in. in diameter are used to make the joint. The basic stress is 275 lb. per sq. in. and the L/D ratio is $\dfrac{5.5}{0.875} = 6.28$. For this ratio Chart 7 gives a safe stress of 274 lb. per sq. in. The diameter factor is 1.33.

Safe load for one bolt $= 274 \times 1.33 \times 5.5 \times 0.875 = 1750$ lb.
Safe load for four bolts $= 1750 \times 4 = 7000$ lb.

Table 15. Standard Bolt Sizes

[A.I.S.C. Manual of Steel Construction]

Dia. of Bolt (in.)	Series	Head: Hexagon Diameter (in.) Long	Head: Hexagon Diameter (in.) Short	Head: Height (in.)	Head: Square Diameter (in.) Long	Head: Square Diameter (in.) Short	Dia. of Bolt (in.)	Series	Nut: Hexagon Diameter (in.) Long	Nut: Hexagon Diameter (in.) Short	Nut: Height (in.)	Nut: Square Diameter (in.) Long	Nut: Square Diameter (in.) Short
1/4	American Standard Regular	7/16	3/8	3/16	1/2	3/8	1/4	American Standard Regular	1/2	7/16	1/4	5/8	7/16
3/8		5/8	9/16	1/4	3/4	9/16	3/8		11/16	5/8	5/16	7/8	5/8
1/2		7/8	3/4	5/16	1	3/4	1/2		15/16	13/16	7/16	1 1/8	13/16
5/8		1 1/16	15/16	7/16	1 5/16	15/16	5/8		1 1/8	1	9/16	1 3/8	1
3/4		1 5/16	1 1/8	1/2	1 9/16	1 1/8	3/4		1 5/16	1 1/8	11/16	1 9/16	1 1/8
7/8		1 1/2	1 5/16	9/16	1 13/16	1 5/16	7/8		1 1/2	1 5/16	3/4	1 13/16	1 5/16
1		1 11/16	1 1/2	5/8	2 1/16	1 1/2	1		1 11/16	1 1/2	7/8	2 1/16	1 1/2
1 1/8		1 15/16	1 11/16	3/4	2 5/16	1 11/16	1 1/8		1 15/16	1 11/16	1	2 5/16	1 11/16
1 1/4		2 1/8	1 7/8	13/16	2 9/16	1 7/8	1 1/4		2 1/8	1 7/8	1 1/8	2 9/16	1 7/8
1 3/8		2 3/8	2 1/16	15/16	2 13/16	2 1/16	1 3/8		2 3/8	2 1/16	1 1/4	2 13/16	2 1/16
1 1/2		2 9/16	2 1/4	1	3 1/16	2 1/4	1 1/2		2 9/16	2 1/4	1 5/16	3 1/8	2 1/4
1 3/8	American Standard Regular	2 3/8	2 1/16	15/16	2 13/16	2 1/16	1 3/8	American Standard Heavy	2 1/2	2 3/16	1 3/8	3	2 3/16
1 1/2		2 9/16	2 1/4	1	3 1/16	2 1/4	1 1/2		2 11/16	2 3/8	1 1/2	3 1/4	2 3/8
1 5/8		2 3/4	2 7/16	1 1/16	3 3/8	2 7/16	1 5/8		2 15/16	2 9/16	1 5/8	3 1/2	2 9/16
1 3/4		3	2 5/8	1 3/16	3 5/8	2 5/8	1 3/4		3 1/8	2 3/4	1 3/4	3 3/4	2 3/4
1 7/8		3 3/16	2 13/16	1 1/4	3 7/8	2 13/16	1 7/8		3 3/8	2 15/16	1 7/8	4 1/16	2 15/16
2		3 7/16	3	1 5/16	4 1/8	3	2		3 9/16	3 1/8	2	4 5/16	3 1/8
2 1/4		3 7/8	3 3/8	1 1/2	4 5/8	3 3/8	2 1/4		4	3 1/2	2 1/4	4 13/16	3 1/2
2 1/2		4 1/4	3 3/4	1 11/16	5 1/8	3 3/4	2 1/2		4 7/16	3 7/8	2 1/2	5 5/16	3 7/8
2 3/4		4 11/16	4 1/8	1 13/16	5 11/16	4 1/8	2 3/4		4 7/8	4 1/4	2 3/4	5 13/16	4 1/4
3		5 1/8	4 1/2	2	6 3/16	4 1/2	3		5 1/4	4 5/8	3	6 3/8	4 5/8
3 1/4		5 9/16	4 7/8	2 3/16	6 11/16	4 7/8	3 1/4		5 11/16	5	3 1/4	6 7/8	5
3 1/2		6	5 1/4	2 5/16	7 3/16	5 1/4	3 1/2		6 1/8	5 3/8	3 1/2	7 3/8	5 3/8
3 3/4		6 7/16	5 5/8	2 1/2	7 3/4	5 5/8	3 3/4		6 9/16	5 3/4	3 3/4	7 7/8	5 3/4
4		6 7/8	6	2 11/16	8 1/4	6	4		7	6 1/8	4	8 7/16	6 1/8
4 1/4		7 1/4	6 3/8	2 13/16	8 3/4	6 3/8	4 1/4		7 7/16	6 1/2	4 1/4	8 15/16	6 1/2
4 1/2		7 11/16	6 3/4	3	9 1/4	6 3/4	4 1/2		7 13/16	6 7/8	4 1/2	9 7/16	6 7/8
4 3/4		8 1/8	7 1/8	3 3/16	9 13/16	7 1/8	4 3/4		8 1/4	7 1/4	4 3/4	9 15/16	7 1/4
5		8 9/16	7 1/2	3 5/16	10 5/16	7 1/2	5		8 11/16	7 5/8	5	10 1/2	7 5/8
5 1/4		9	7 7/8	3 1/2	10 13/16	7 7/8	5 1/4		9 1/8	8	5 1/4	11	8
5 1/2		9 3/8	8 1/4	3 11/16	11 5/16	8 1/4	5 1/2		9 9/16	8 3/8	5 1/2	11 1/2	8 3/8
5 3/4		9 13/16	8 5/8	3 13/16	11 13/16	8 5/8	5 3/4		10	8 3/4	5 3/4	12	8 3/4
6		10 1/4	9	4	12 3/8	9	6		10 3/8	9 1/8	6	12 1/2	9 1/8

In calculating the bearing stress at an angle to the grain the Hankinson formula, given in Article 22, is recommended. The joint shown in Figure 9 will serve as an example to illustrate the method used in calculating the safe load for a joint when the load is applied at an angle to the grain.

The center member is 6 in. thick and the side members are 3 in. thick; the joint consists of seasoned southern pine to be used in a dry inside location. Four common bolts $\frac{7}{8}$ in. in diameter are used to make the joint. The joint is the same size as that used in the previous examples in which the safe stress parallel to the grain was 660 lb. per sq. in. and the safe stress perpendicular to the grain was 274 times 1.33, or 364 lb. per sq. in.

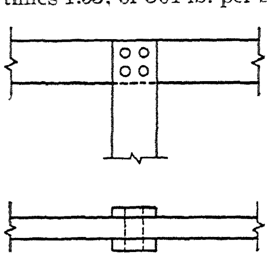


Fig. 8. Bolted joint with load perpendicular to grain.

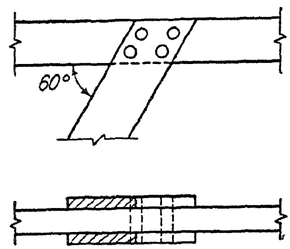


Fig. 9. Bolted joint with load at an angle to grain.

The sine² of 60° is 0.75 and the cosine² of 60° is 0.25. The safe stress to use when the angle between the direction of load and the direction of grain is 60° then becomes

$$N = \frac{660 \times 364}{660 \times 0.75 + 364 \times 0.25} = 410 \text{ lb. per sq. in.}$$

Safe load for four bolts = $410 \times 4 \times 5.5 \times 0.875 = 7880$ lb.

39. Design Details. When joints are exposed to the weather use three-fourths of the values obtained from the procedure outlined in the previous article; and if wet or damp most of the time, use two-thirds. For green lumber that is allowed to season under load, use 40 per cent of the calculated safe load for dry lumber.

If the side members in a three-member joint are thicker than one-half the thickness of the main member, the safe load for the joint is determined in the same manner as for a joint with side members one-half the thickness of the main member. However, if the side members are less than one-half the thickness of the main member, consider the length of the bolt as twice the thickness of the thinnest side member used. As an example, for 3-in. side members and an 8-in. main member, use 6 in.

When a joint consists of two members of equal thickness, calculate the safe load by considering a main member twice as thick as one of the members and reducing the result by 50 per cent. If the members in a

two-member joint are of unequal thickness, calculate the safe load based on a piece twice the thickness of the thinnest member and reduce this safe load by 50 per cent. If a joint consists of four members of equal

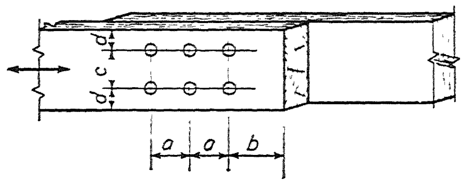


FIG. 10. Method of measuring spacing parallel to grain.

a = Center to center spacing. Minimum four times bolt diameter.

b = End distance. In tension, 7 times bolt diameter for softwoods and 5 times bolt diameter for hardwoods. In compression, 4 times bolt diameter.

c = Row spacing. Determined by net area at critical section. This area must be at least 80 per cent of total area in bearing under all the bolts.

d = Edge distance. At least 1½ times bolt diameter for L/D ratios of 5 or 6. For ratios greater than 6 increase slightly and for ratios less than 5 decrease slightly. A ratio of 2 : 1 between row spacing and edge distance is preferable.

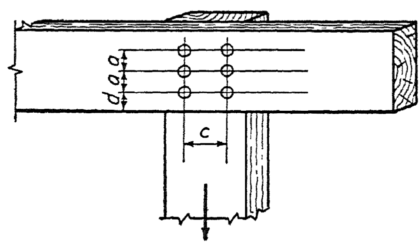


FIG. 11. Method of measuring spacing perpendicular to grain.

a = Center to center spacing—4 times bolt diameter, unless design load is much less than bolt bearing capacity of side members. In this case spacing may be reduced. When metal splice plates are used, spacing required is sufficient space to tighten nuts.

b = No end distance in this type of joint.

c = Row spacing, minimum—2½ times bolt diameter for L/D ratios of 2, and 5 times bolt diameter for L/D ratios of 6 or greater.

d = Edge distance, minimum—4 times bolt diameter. In compression the edge distance would be at the opposite edge from that shown in Figure 11.

thickness, the safe load to be used is one and one-half times the calculated safe load for a piece the thickness of one of the members.

Figures 10 and 11 illustrate the method of measuring the spacings and the end and edge distances.

The maximum efficiency of a joint when the load is parallel to the grain is obtained when the L/D ratio is 6 or more. For loads perpendicular to the grain, the maximum efficiency occurs at a L/D ratio of 8 and decreases for smaller or larger L/D ratios.

Staggering bolts should be avoided when the load acts parallel to the grain. If an odd bolt is required, the usual practice is to alter the diameter of all the bolts so that an even number can be used. Moreover, if an odd bolt is used and placed in the center of the timber, it must be considered as being between the last pair of bolts in computing the critical section, or it must be placed at a distance from them greater than the center-to-center spacing. For loads acting perpendicular to the grain, staggering is preferable to avoid splitting.

Seasoning after assembly may start splitting at the bolts, and for this reason cross bolts are recommended. The tests conducted at the Forest Products Laboratory have not been sufficient to furnish rules for the size and number of cross bolts to use under various conditions. However, cross bolts can be smaller in diameter than the main bolts in a joint and in most cases only one or two are required. In a tension joint the cross bolts should be placed in the end distance, and in a joint with the load perpendicular to the grain, cross bolts should be placed in the main member just outside the main bolt group.

It is impossible to present general rules covering the spacing and end and edge distances for bolts when the load is applied at an angle to the grain. It is important, however, to arrange the bolts so that the center of resistance of the bolt group passes through the intersection of the gravity axis of the members.

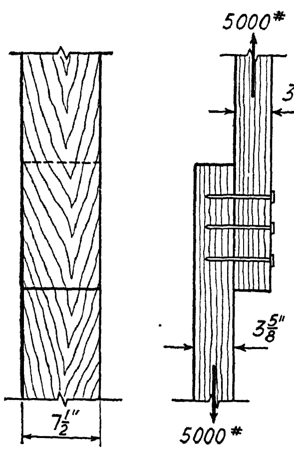


FIG. 12. Tension lap joint.

Of particular interest to the designing engineer using bolts should be the subject of camber. As a general rule bolts are inserted into holes slightly larger than their diameter, which naturally increases the yield of a timber structure. For this reason the engineer should provide ample cambers for bolted structures and should keep to a minimum the number of joints and splices.

ILLUSTRATIVE PROBLEMS

Problem. Find the number and size of lag screws and bolts for the tension joint in Figure 12. Use seasoned shortleaf southern pine.

Lag Screws: The penetration into the member receiving the point should be $9D$. Select a screw with a diameter of ⅜ in. This makes the total length $= 9 \times \frac{3}{8} + 3.625 = 7$ in. For this length the threaded portion of the screw, from Table 11, is 4 in. This leaves a shank 3 in. long, which makes it necessary to reduce the

general equation because the shank does not extend to the plane of contact between the two members.

The percentage to use as a multiplier in the equation may be found as follows:

$$\text{Percentage} = 100 - 20\,\frac{(t_1 + t_2 - L_1)}{t_2}$$

where t_1 = thickness of washer,
t_2 = thickness of cleat,
L_1 = length of shank.

$$\text{Percentage} = 100 - 20\left(\frac{0.0781 + 3.625 - 3}{3.625}\right) = 96.2 \text{ per cent.}$$

The ratio of cleat thickness to the diameter of screw is $\dfrac{3.625}{0.375} = 9.7$, and according to Chart 3 the safe load can be increased by 25 per cent.

Safe load: $P = 1900 \times 0.962 \times 1.25 \times 0.375^2 = 321$ lb.

No. of lag screws required $= \dfrac{5000}{321} = 16$

Bolts: The length of bolt for calculating the safe load is $2 \times 3.625 = 7.25$. Using bolts with a diameter of 1¼ in. the L/D ratio is 5.8; for this L/D ratio Chart 5 shows a working stress of 885 lb. per sq. in.

Safe load $= 885 \times 0.8 \times 0.5 \times 1.25 \times 7.25 = 3210$ lb.

No. of bolts required $= \dfrac{5000}{3210} = 2$

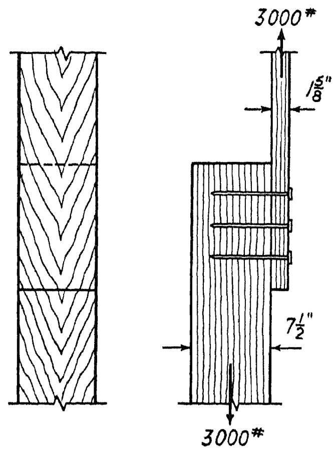


FIG. 13. Tension lap joint.

Problem. Find the number and size of lag screws and bolts for the lap joint in Figure 13. Use seasoned shortleaf southern pine.

Lag Screws: Use ½-in. lag screws with a length of 6½ in. For this length the shank is 3 in. and extends into the 8-in. timber 1⅜ in. The ratio of the length of shank in the 8-in. member to the diameter is 2.75, which makes it possible to increase the load by 24 per cent (Chart 4).

The ratio of cleat thickness to the diameter is 3.25, so it is necessary to reduce the load by 4 per cent (Chart 3).

Safe load for one screw $= 1900 \times 1.24 \times 0.96 \times 0.5^2 = 566$ lb.

No. of screws required $= \dfrac{3000}{566} = 6$

Bolts: The length of the bolt for calculating the safe load is $2 \times 1.625 = 3.25$. Use ½-in. bolts. $L/D = 6.5$.

$$\text{Safe load for one bolt} = 810 \times 0.8 \times 0.5 \times 3.25 \times 0.5 = 525 \text{ lb.}$$

$$\text{No. of bolts required} = \frac{3000}{525} = 6$$

Remarks: A comparison of this problem with the previous one brings out the fact that the thickness of cleat used has a bearing on the load-carrying capacity of lag screws. It is preferable to use larger diameters when thick cleats are used in the joint, making the ratio of cleat thickness to the diameter of the lag screw approximately 3.5.

Problem. Design a bolted tension joint to carry 55,000 lb. Use Douglas fir (coast region) and assume that the lumber is seasoned and the joint is to be used in a dry inside location.

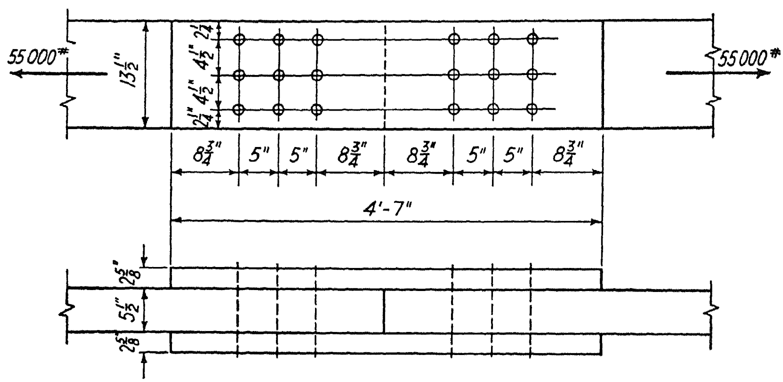


FIG. 14. Detail of bolted tension joint.

Bolts: Assume a main member 6 in. thick with 3-in. splice plates. Try 1¼-in. bolts. $L/D = \dfrac{5.5}{1.25} = 4.4$.

For an L/D ratio of 4.4 the working stress (Chart 5) is 1130 lb. per sq. in.
Safe stress with wood splice plates $= 1130 \times 0.8 = 904$ lb. per sq. in.
Safe load for one bolt $= 904 \times 1.25 \times 5.5 = 6220$ lb.

$$\text{No. of bolts required} = \frac{55{,}000}{6220} = 9$$

Try a 6 by 14-in. member with area $= 74.25$ sq. in. Bearing under all bolts $= 9(1.25 \times 5.5) = 61.9$ sq. in. Required area equals 80 per cent of bearing area under all bolts plus bearing area of number of bolts in a row.

80 per cent of bearing area $= 61.9 \times 0.8$	$= 49.5$
With 3 bolts in a row $= 3(1.25 \times 5.5)$	$= \underline{20.6}$
Required area	$= 70.1$ sq. in.

A 6 by 14-in. member will satisfy.

Problem. Find the size and number of bolts required for a two-member joint with a 7000 lb. load acting in tension at an angle of 60°. Use dry shortleaf southern pine with metal splice plates. Detail the joint.

Assumptions: Members will be 4 in. thick, and bolts will have a diameter of ½ in.

L/D ratio: $L = 3.625.$ $L/D = \dfrac{3.625}{\frac{1}{2}} = 7.25$

Safe load for one bolt

Parallel to grain: $715 \times 0.5 \times 3.625 = 1295$ lb.

Perpendicular to grain: $260 \times 1.68 \times 0.5 \times 3.625 = 795$ lb.

At angle of 60°: $\dfrac{1295 \times 795}{1295 \times 0.75 + 795 \times 0.25} = 880$ lb.

Member A:

No. of bolts required $= \dfrac{7000}{880} = 8$

Member B:

No. of bolts required $= \dfrac{7000}{1295} = 6$

Size of Member B. Consider 2 rows of bolts.

80 per cent of bearing under all bolts $= 0.8 \times 6(0.5 \times 3.625)$	=	8.7
Bearing area of 2 bolts at critical section $= 2(0.5 \times 3.625)$	=	3.625
Required area	=	12.325 sq. in.

A 4 by 4-in. member with an area of 13.14 sq. in. will satisfy.

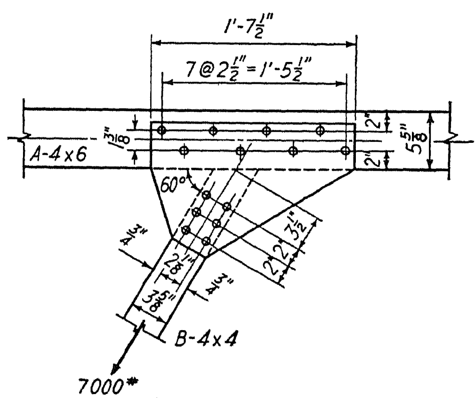


Fig. 15. Detail of joint.

Problem. Find the size and number of lag screws for the joint shown in Figure 16. Use seasoned Douglas fir.

Ratio for bolts with L/D of 12 (see Article 32).

Perpendicular to grain:

Let df = diameter factor

Safe load $= 140 \times df \times A$

Parallel to grain:

Safe load $= 344 \times A$

$$\text{Ratio} = \frac{140 \times df \times A}{344 \times A} = 0.41df$$

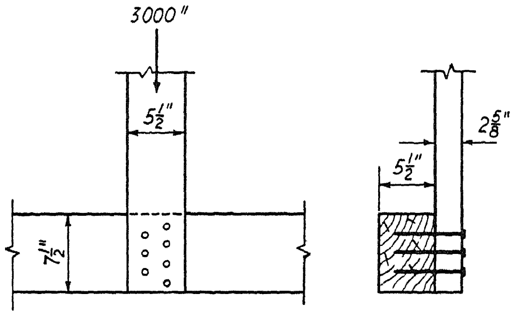


FIG. 16. Lag screw joint.

Lag screws: Use ½-in. lag screws.

Length $= 9 \times \frac{1}{2} = 4.5 + 2.625 = 7.125$ in. Use 7.5 in.

Ratio of depth of shank in main member to diameter of screw $= \dfrac{0.875}{0.5} = 1.75$.

This allows an increase in safe load of 15 per cent.

Ratio of cleat thickness to diameter of screw $= \dfrac{2.625}{0.5} = 5.25$

This allows an increase in safe load of 19 per cent

Safe load parallel to grain $= 1900 \times 1.15 \times 1.19 \times 0.5^2 = 650$ lb.

Safe load perpendicular to grain $= 650 \times 0.41df = 650 \times 0.41 \times 1.68 = 448$ lb.

No. of lag screws required $= \dfrac{3000}{448} = 7$

CONNECTORS

40. General. Probably no single factor is more responsible for revolutionizing timber design than the development of modern timber connectors. The results of tests on many types of connectors were first reported in this country in 1933 by Nelson S. Perkins, Peter Landsem, and G. W. Trayer. These tests formed the basis for assigning safe working loads to connectors when used with native woods. In 1934 the Timber Engineering Company, of Washington, D. C., subsidiary of the National Lumber Manufacturer's Association, acquired the patent rights on a number of these connectors for the purpose of distributing

them on a commercial basis. Since then many additional tests and refinements in manufacturing have been made.

Timber connectors are essentially metal rings or plates that are embedded partly in each face of adjacent members to transmit the load from one to the other. They are used in combination with bolts of small diameter. Connectors enlarge the bearing area of the joint stress, thereby making it possible in many cases to develop the full allowable load of the members connected.

41. Types of Connectors. More than 60 different types of connectors have been patented in the United States and Europe. Among the most commonly used in this country are the split ring, toothed ring, shear and claw plates, spike grids, and clamping plates.

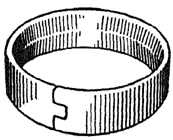


FIG. 17. Split ring.

Manufactured from low carbon steel: Used between two timber faces for heavy construction and fits into pre-cut grooves in the timber faces. The tongue and groove "split" permits simultaneous ring bearing against the core wall and outer wall of the groove into which the ring is placed. The inside bevel and mill edge facilitate installation into and removal from its groove.

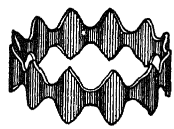


FIG. 18. Toothed ring.

Manufactured from low carbon steel: Used between two timber faces for comparatively light construction and embedded into the contact faces of the joint members by means of pressure.

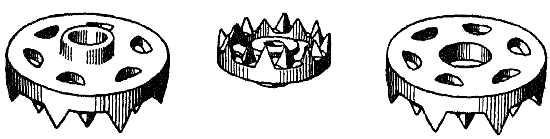


FIG. 19. Claw plates.

Malleable iron connectors: Used as "units" either in pairs for timber-to-timber connections or singly in making timber-to-metal connections. The female plates are adapted to use when the connector must lie flush with the surface of the timber. Claw plates are installed by forcing the teeth into the wood beyond the depth of the circular dap cut to receive the rim and plate portions.

42. Sizes. The connectors illustrated in the previous article are manufactured in several different sizes. The split and toothed rings, claw

plates, and shear plates are round, and their size is specified by diameter. The spike grids are square, and the length of one side of the square

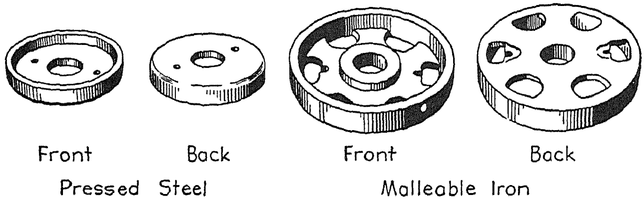


FIG. 20. Shear plates.

These plates when installed lie flush with the timber surface. They are used as "units" in pairs for timber-to-timber joints with two plates placed back to back, or singly in timber-to-metal joints with the plate placed with its back toward the metal. The plates fit into pre-cut grooves in the timber faces.

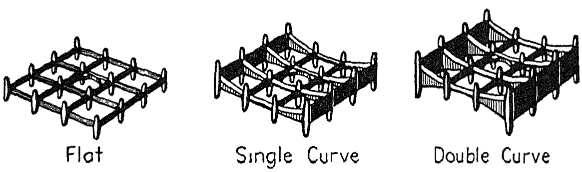


FIG. 21. Spike grids.

Manufactured from malleable cast iron: Used primarily in pier and trestle construction between either flat or curved surfaces. They are embedded into the wood surfaces by means of pressure.

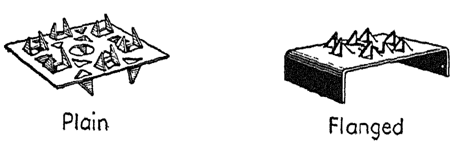


FIG. 22. Clamping plates.

Stamped from metal sheets: Used as railroad "tie spacers" between ties and guard timbers to keep the ties properly spaced, or where timbers overlap at right angles. The plain clamping plate with teeth on opposite faces is seated by means of a special block which protects the connector during the driving process. The flanged clamping plate with teeth on one face only is driven into place with a maul or ram, the connector being protected from damage by a steel cover plate.

gives the size. Clamping plates are manufactured with and without flanges. Those without flanges are usually square and those with flanges are rectangular. Table 16 lists the dimensions of the types of connectors most commonly used.

Table 16. Connector Sizes

[Dimensions in inches]

Timber Connector	Shipping Wt (lb per 100)	Size	Dimensions of Metal		Groove Size		
			Depth	Thickness	Inside Diameter	Width	Depth
Split rings	30	2½	0.75	0.163	2.56	0.18	0.37
	76	4	1.00	.193	4.08	.21	.50
	96	6	1.25	.245	6.12	.27	.62
Toothed rings	9	2	.94	.061			
	12	2⅝	.94	.061			
	15	3⅜	.94	.061			
	18	4	.94	.061			
Claw plates	58	2⅝	.75		2.14	.25	.38
	70	3⅛	.75		2.66	.25	.38
	93	4	.75		3.57	.25	.38
Shear plates: Pressed steel	40	2⅝	.375	.169	2.25	.19	.45
Shear plates: Malleable iron	100	4	.62	.20	3.49	.27	.64
Spike grids: Flat	50	4⅛ x 4⅛	1.00				
Spike grids: Single curve	75	4⅛ x 4⅛	1.38				
Spike grids: Double curve	107	4⅛ x 4⅛	1.75				
Clamping plates: Plain	65	5¼ x 5¼		.077			
Clamping plates: Flanged	200	5 x 8	2.00	.122			

43. Bolts and Washers. The design loads for connectors are based on a minimum size of bolts and washers. Usually, if the bolt size is increased beyond the minimum, the design load can be increased somewhat. The allowable increases for 4-in. split rings are 3½ per cent and 7 per cent in lumber 2⅝ in. thick and 5½ per cent and 10 per cent in lumber 3 in. thick for ⅞-in. and 1-in. bolts, respectively, when used in place of the ¾-in. bolt. Table 17 lists the minimum bolt and washer sizes to use with the various connectors in order to develop their standard design loads.

44. Installation Data. Connectors require certain minimum widths and thicknesses of lumber, depending upon the size of the connector and whether or not the lumber will have connectors in one face only or in two faces. The standard end distance depends upon the type of stress in the member; i.e., whether it is in tension or compression. Standard edge distance and center-to-center spacing are based on the direction of load or on the angle between the load and the grain of the piece.

Table 17. Sizes of Bolts and Washers Used with Connectors

[Dimensions in inches]

Timber Connector	Size	Bolt	Round Washers: Cast or Malleable Iron: Diameter	Round Washers: Wrought Iron: Diameter	Round Washers: Wrought Iron: Thickness	Plate Washers: Size	Plate Washers: Thickness
Split rings	2½	½	2⅛	1⅜	3/32	2	⅛
	4	¾	3	2	5/32	3	3/16
	6	¾	3			3	¼
Toothed rings	2	½	2			2	3/16
	2⅝	⅝	2⅝			2½	¼
	3⅜	¾	3			3	¼
	4	¾	3½			3½	⅜
Claw plates: Male	2⅝	½	2⅛	2	5/32	2	3/16
Claw plates: Female	2⅝	⅞	3½	2	5/32	3	¼
Claw plates: Male	3⅛	½	2⅛	2	5/32	2	3/16
Claw plates: Female	3⅛	⅞	3½	2	5/32	3	¼
Claw plates: Male	4	¾	3	4	11/64	3	⅜
Claw plates: Female	4	1¼	4⅜	4	11/64	4	⅜
Shear plates: Pressed steel	2⅝	¾	3	2	5/32	3	¼
Shear plates: Malleable iron	4	¾	3	2	5/32	3	¼
Shear plates: Malleable iron	4	⅞	3½	2¼	11/64	3	¼
Spike grids: Flat	4⅛ x 4⅛	¾ or 1	3 or 4			3	⅜
Spike grids: Single curve	4⅛ x 4⅛	¾ or 1	3 or 4			3	⅜
Spike grids: Double curve	4⅛ x 4⅛	¾ or 1	3 or 4			3	⅜
Clamping plates: Plain	5¼ x 5¼	¾	3			3	¼
Clamping plates: Flanged	5 x 8	¾	3			3	¼

Designs of most joints include more connectors than necessary to develop the load carried. This is true because the load on the joint divided by the safe design load of the connector will rarely require an exact number of connectors. Moreover, in multiple joints it is often necessary to use several more connectors than required to carry the load in order to keep the joint symmetrically loaded. For these reasons it is possible to reduce spacings and end and edge distances for most connectors. Minimums have been determined which are based on a reduction in the standard design loads. In the design of joints requiring more than one group of connectors it is possible to reduce the spacing and the end distance. However, the percentage used in reducing the spacing should be the same as that used in reducing end distance.

Table 18. Installation Data for Connectors [Dimensions in inches]

Timber Connector		Size	Minimum Dimensions of Timber: Width	Thickness: Connectors in One Face	Thickness: Connectors in Two Faces	End Distance, Tension: Standard	End Distance, Tension: Minimum Reduction in Load, 37.5%	End Distance, Compression: Standard	End Distance, Compression: Minimum Reduction in Load, 37.5%	Edge Distance 0°–30°: Standard and Minimum	Edge Distance 30°–90°: Standard Compression Side	Edge Distance 30°–90°: Minimum Compression Side Reduction in Load, 15%	Edge Distance 30°–90°: Standard and Minimum Opposite Compression Side	Spacing 0°–30° Parallel: Standard	Spacing 0°–30° Parallel: Minimum Reduction in Load, 50%	Spacing 0°–30° Perpendicular: Standard and Minimum	Spacing 30°–90° Parallel: Standard and Minimum	Spacing 30°–90° Perpendicular: Standard	Spacing 30°–90° Perpendicular: Minimum Reduction in Load, 15%
Split rings		2½	3⅝	1	1⅝	5½	2¾	4	2½	1¾	2¾	1¾	1¾	6¾	3⅜	3½	3½	4½	3½
		4	5½	1	2	7	3½	5½	3¾	2¾	3¾	2¾	2¾	9	4⅞	5½	5½	6½	5½
		6	7½	1⅝	2	9	4½	7½	4¼	3¾	4¾	3¾	3¾	12	7	7½	7½	8½	7½
Claw plates	Male	2⅝	3⅝	1⅝	2	5½	2¾	4	3	1¾	2¾	1¾	1¾	6¾	3⅜	3½	3½	4½	3½
	Female	2⅝	3⅝	1⅝	2	5½	2¾	4	3	1¾	2¾	1¾	1¾	6¾	3⅜	3½	3½	4½	3½
	Male	3⅛	4⅝	1⅝	2	6	3	4½	3½	2⅛	3⅛	2⅛	2⅛	7¾	4	4¼	4¼	5¼	4¼
	Female	3⅛	4⅝	1⅝	2	6	3	4½	3½	2⅛	3⅛	2⅛	2⅛	7¾	4	4¼	4¼	5¼	4¼
	Male	4	5½	1⅝	2	7	3½	5½	4½	2¾	3¾	2¾	2¾	9	4⅞	5½	5½	6½	5½
	Female	4	5½	1⅝	2	7	3½	5½	4½	2¾	3¾	2¾	2¾	9	4⅞	5½	5½	6½	5½
Shear plates	Pressed steel	2⅝	3⅝	1	1⅝	5½	2¾	4	2½	1¾	2¾	1¾	1¾	6¾	3⅜	3½	3½	4½	3½
	Malleable iron	4	5½	1⅝	1¾	7	3½	5½	3¼	2¾	3¾	2¾	2¾	9	4⅞	5½	5½	6½	5½
	Malleable iron	4	5½	1⅝	1¾	7	3½	5½	3¼	2¾	3¾	2¾	2¾	9	4⅞	5½	5½	6½	5½
											Standard	Min. Reduce Load, 15%							
Toothed rings		2	3	1	1⅝	3½	2	2		1½	2	1½		3½	3	2½	2½	3	
		2⅝	3⅝	1	1⅝	4½	2⅝	2⅝		1¾	2½	1¾		4½	3⅝	3⅛	3⅛	3½	
		3⅜	4⅝	1	1⅝	5¾	3⅜	3⅜		2¼	3	2¼		5¾	4⅜	3⅞	3⅞	4½	
		4	5½	1	1⅝	7	4	4		2¾	3½	2¾		7	5	4½	4½	5½	
						Standard		Min. Reduce Load, 15%											
Spike grids	Flat	4⅛ x 4⅛	5½	1⅝	2	7		5		3¾	3¾	2¾		7		5½	5½	5½	
	Single curve	4⅛ x 4⅛	5½	1⅝		7		5		3¾	3¾	2¾		7		5½	5½	5½	
	Double curve	4⅛ x 4⅛				7		5		3¾	3¾	2¾		7		5½	5½	5½	
Clamping plates	Plain	5¼ x 5¼	6½	1⅝	2⅝	5				3¼	3¼			6		6	6	6	
	Flanged	5 x 8	6½	2⅝	5½	5				3¼	3¼			8½		5½	8½	5½	

Figure 23 shows the method of measuring spacing and end and edge distances for connectors.

In Table 18 standard and minimum edge distances are listed for the compression side of the ring. The compression side of the ring is deter-

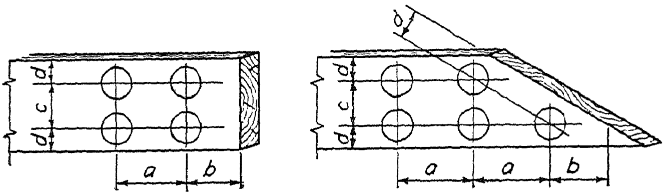


FIG. 23. Connector spacings, end and edge distances.

a = Connector spacing parallel to grain.
b = End distance.
c = Connector spacing perpendicular to grain.
d = Edge distance.

mined by the direction of the load and is measured from the center of the ring to the edge of the piece upon which the load is acting. It is important to consider the compression side of the ring because it is this edge distance that usually determines the width of the member that has the load acting at an angle to its grain.

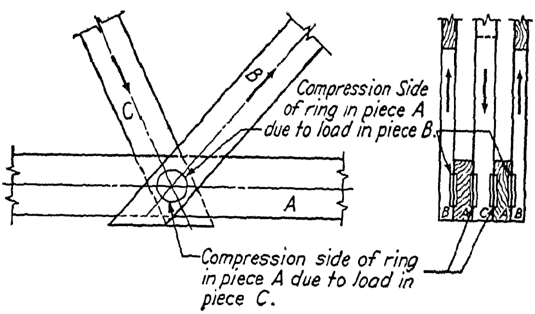


FIG. 24. Compression side of ring.

45. Safe Loads. The safe working loads for connectors are based on the species and grade of lumber used. These are divided into three general groups, A, B, and C. Group A includes dense structural grades of Douglas fir and southern pine. Group B contains non-dense structural grades of Douglas fir and southern pine and structural grades of western larch, tamarack, ash, beech, birch, maple, and oak. Group C includes structural grades of cypress and redwood.

The safe loads apply only when the connectors are used in seasoned lumber which has a moisture content not exceeding 15 per cent within ½ in. of the surface. For connectors used in green lumber (about 24 per cent moisture content) use 67 per cent of the safe loads for split rings, claw plates, and shear plates and 60 per cent of the safe loads for toothed rings, spike grids, and clamping plates, with the exception of the flanged clamping plate, which has the same working load for green or dry material. For moisture contents between 15 and 24 per cent, the percentage can be determined by interpolation.

Wood can support loads that will remain on a structure for a short time greatly in excess of permanently applied loads, and for this reason increases in the safe design loads for wind or earthquake loads are permissible providing the resulting size or number of connectors is not less than required for dead and live loads alone. For split rings and spike grids the safe loads may be taken as 130 per cent of the standard design loads, and for toothed rings and clamping plates 116 per cent may be used. For claw and shear plates use 116 per cent when the load is parallel to the grain and 130 per cent when the load is perpendicular to the grain.

The load on a split ring due to a force producing impact may be taken as 57.5 per cent of the sum of the force as a static load and the load due to its impact, but the load shall not be considered less than the load caused by the force if it were acting as a static load. For all other connectors use 115 per cent of the sum of the force as a static load and the load due to its impact.

Charts 9, 10, and 11 represent the standard design loads for split ring connectors. Values for other connectors may be obtained from the *Manual of Timber Connector Construction,* published by the Timber Engineering Company.

46. Net Section. The unit working stresses for structural lumber permit certain knot sizes in each grade, which means that the net area of a timber is actually the gross area minus the projected area of the knot. However, it is not necessary to deduct the cross-sectional area of the knot in design because the working stresses have been reduced from the basic stresses determined for clear material and allowances made for the maximum-size knot permitted in the grade. For this reason it is not necessary to check the net section of a joint with connectors for the lower structural grades of lumber providing, of course, that a knot of maximum size permitted in the grade will not occur at the plane of the critical section. For the higher structural grades the knot limitations are more severe, and when these grades are used it is best to calculate the net section.

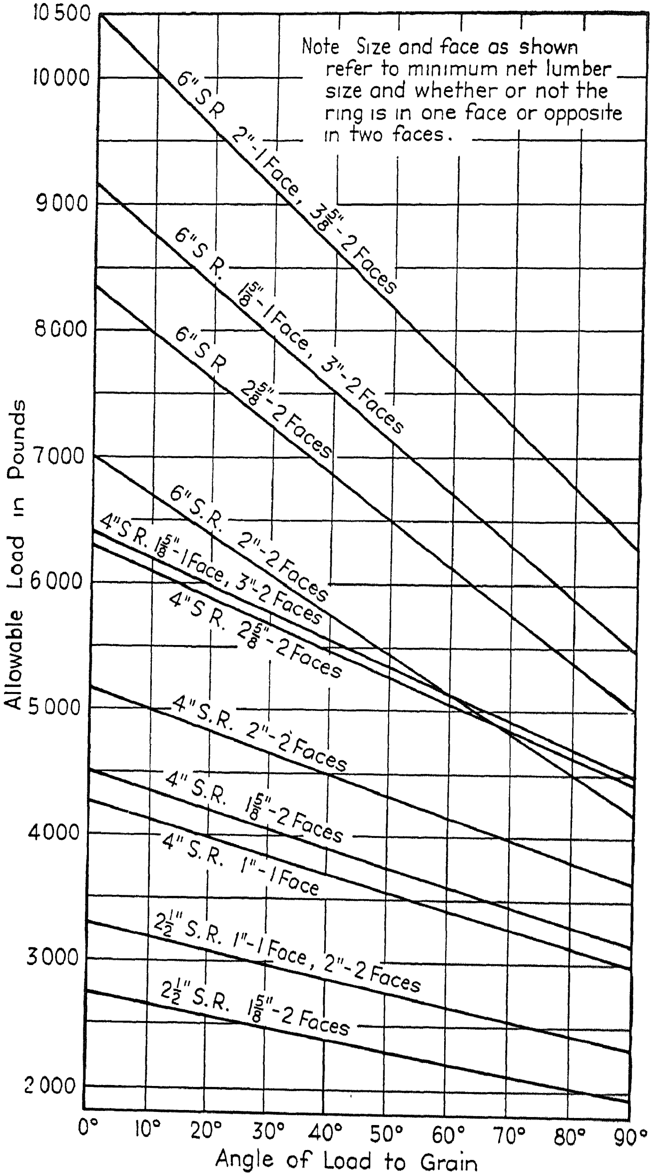


CHART 9. Safe loads—split rings.

GROUP A. Dense structural grades of southern pine and Douglas fir.

Loads given are for a two-member joint assembly with one split ring, a bolt, two washers, and a nut.

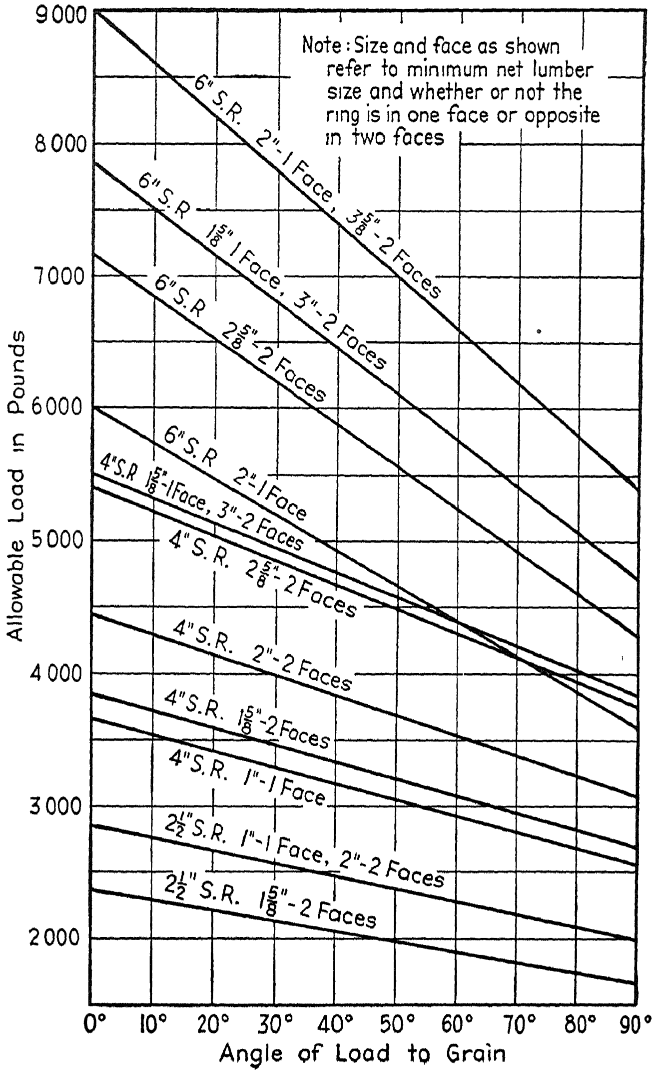


CHART 10. Safe loads—split rings.

GROUP B. Non-dense structural grades of southern pine and Douglas fir, structural grades of western larch, tamarack, ash, beech, birch, maple, and oak.

Loads given are for two-member joint assembly with one split ring, a bolt, two washers, and a nut.

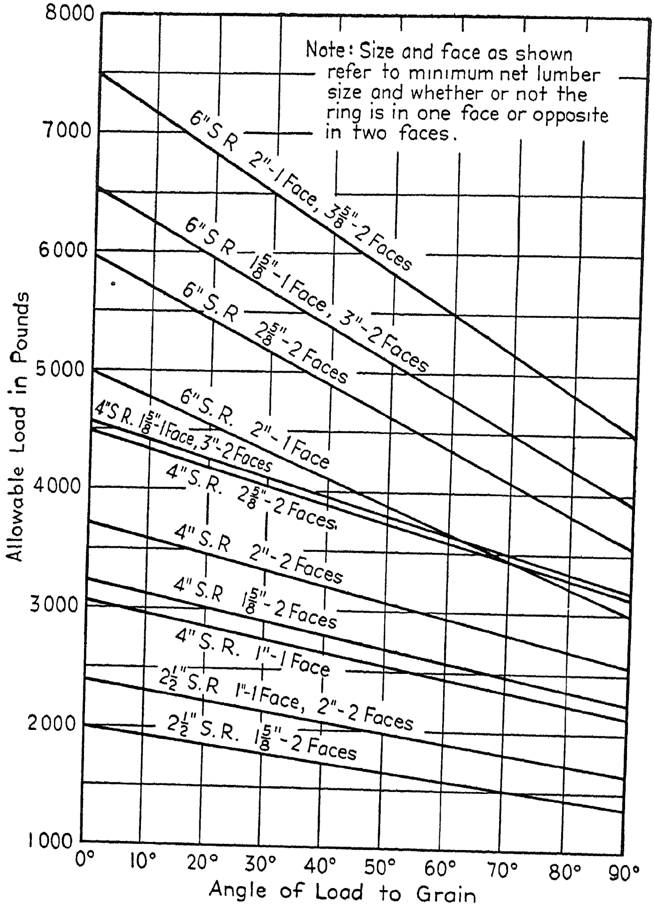


CHART 11. Safe loads—split rings.

GROUP C. Structural grades of cypress and redwood.

Loads given are for a two-member joint assembly with one split ring, a bolt, two washers, and a nut.

Table 19, prepared by the Timber Engineering Company, is based on recommendations of the Forest Products Laboratory. The constants may be used in determining the net section required. The net cross-sectional area necessary at the critical section is obtained by selecting the appropriate constant for the type of loading under consideration and multiplying it by the total load.

Table 19. Constants for Use in Determining Required Net Section

Type of Loading	Thickness of Wood Member	Constants for Each Connector Load Group		
		Group A	Group B	Group C
Standard	4″ or less	0.00041	0.00046	0.00048
	Over 4″	.00051	.00058	.00060
Wind or earthquake	4″ or less	.00031	.00036	.00037
	Over 4″	.00039	.00044	.00046
Impact	4″ or less	.00023	.00027	.00028
	Over 4″	.00029	.00033	.00035
Dead load	4″ or less	.00047	.00053	.00055
	Over 4″	.00058	.00067	.00069

47. Action of a Split Ring. The split ring connector, as the name signifies, has a tongue and slot split in its circumference. The purpose of this split is to provide simultaneous bearing against the core of wood inside the ring and against the wood outside the ring.

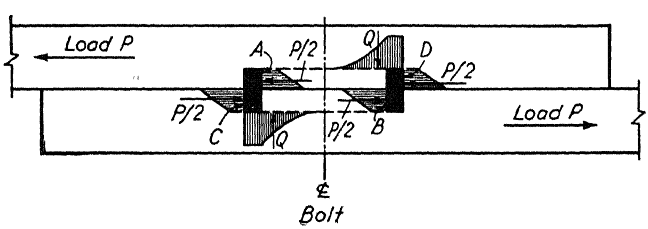


FIG. 25. Theoretical action of a split ring under load.

The core of wood inside the ring is made slightly larger than the diameter of the ring so that as soon as the ring is installed there is bearing at points A and B. As the groove in the wood is made slightly larger than the thickness of the ring, a slight slip in the joint will occur

Table 20. Projected Areas of Connectors and Bolts

[Manual of Timber Connector Construction, Timber Engineeiing Co., 1939.]

Connector No.	Connector Size	Bolt Diam. (inches)	Placement of Connectors	Total Projected Area in Square Inches of Connectors and Bolts in Lumber Thickness of 1⅝″	2″	2⅝″	3″	3⅝″	4″	5½″	6″	7½″	8″
Split Rings													
1	2½	½	One Face	1.73	1.92	2.23	2.42	2.73	2.92	3.67	3.92	4.67	4.92
		½	Two Faces	2.64	2.83	3.14	3.34	3.64	3.83	4.58	4.83	5.58	5.83
2	4	¾	One Face	3.09	3.37	3.84	4.12	4.59	4.87	6.00	6.37	7.50	7.87
		¾	Two Faces	4.97	5.25	5.72	6.00	6.47	6.75	7.16	8.25	9.38	9.75
3	6	¾	One Face	4.91	5.19	5.66	5.94	6.41	6.69	7.10	8.19	9.32	9.69
		¾	Two Faces	.....	8.88	9.35	9.63	10.10	10.38	10.79	11.88	13.01	13.38
Toothed Rings													
1	2	½	One Face	1.52	1.71	2.02	2.21	2.52	2.71	3.46	3.71	4.46	4.71
		½	Two Faces	2.22	2.41	2.72	2.91	3.22	3.41	4.16	4.41	5.16	5.41
2	2⅝	⅝	One Face	1.95	2.18	2.58	2.81	3.20	3.44	4.37	4.69	5.62	5.94
		⅝	Two Faces	2.89	3.12	3.52	3.75	4.14	4.38	5.31	5.63	6.56	6.88
3	3⅜	¾	One Face	2.46	2.74	3.21	3.49	3.96	4.24	5.37	5.74	6.87	7.24
		¾	Two Faces	3.70	3.98	4.45	4.73	5.20	5.48	6.61	6.98	8.11	8.48
4	4	¾	One Face	2.75	3.03	3.50	3.78	4.25	4.53	5.66	6.03	7.16	7.53
		¾	Two Faces	4.28	4.56	5.03	5.31	5.78	6.06	7.19	7.56	8.69	9.06
Claw Plates													
1 and 1A	2⅝	½	One Face	2.41	2.60	2.91	3.10	3.41	3.60	4.35	4.60	5.53	5.60
		½	Two Faces	......	4.19	4.50	4.69	5.00	5.19	5.94	6.19	6.94	7.19
2 and 2A	3⅛	½	One Face	2.78	2.97	3.28	3.47	3.78	3.97	4.72	4.97	5.72	5.97
		½	Two Faces	....	4.93	5.24	5.43	5.74	5.93	6.68	6.93	7.68	7.93
3 and 3A	4	¾	One Face	3.66	3.94	4.41	4.69	5.16	5.44	6.57	6.94	8.07	8.44
		¾	Two Faces	....	6.37	6.84	7.12	7.59	7.87	9.00	9.37	10.50	10.87
Shear Plates													
1	2⅝	¾	One Face	1.92	2.20	2.67	2.95	3.42	3.70	4.83	5.20	6.33	6.70
		¾	Two Faces	2.62	2.90	3.37	3.65	4.12	4.40	5.53	5.90	7.03	7.40
2	4	¾	One Face	3.23	3.51	3.98	4.26	4.73	5.01	6.14	6.51	7.64	8.01
		¾	Two Faces	......	5.52	5.99	6.27	6.74	7.02	8.15	8.52	9.65	10.02
2-A	4	⅞	One Face	3.36	3.69	4.24	4.56	5.11	5.44	6.75	7.19	8.50	8.94
		⅞	Two Faces	.....	5.62	6.17	6.49	7.04	7.37	8.68	8.12	10.43	10.87
Spike Grids													
1, 2 and 3	4⅛ × 4⅛	¾	One Face	2.90	3.18	3.65	3.93	4.40	4.68	5.81	6.18	7.31	7.68
		¾	Two Faces	......	4.87	5.34	5.62	6.09	6.37	7.50	7.87	9.00	9.37
		1	One Face	3.18	3.56	4.18	4.56	5.18	5.56	7.06	7.56	9.00	9.56
		1	Two Faces	......	5.13	5.75	6.13	6.75	7.13	8.63	9.13	10.63	11.13

when the load is applied. When the load is acting on the joint the rin is in bearing at points C and D. Thus wood is brought into bearing o the inside and the outside of the ring.

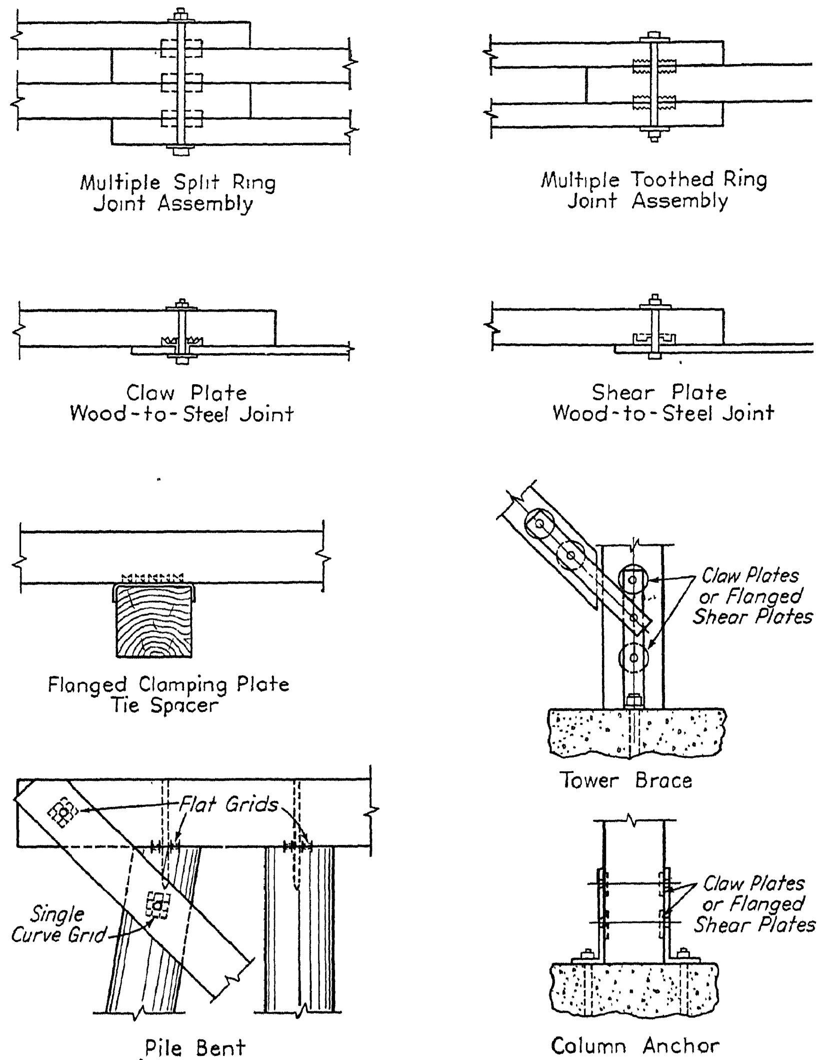


FIG. 26. Typical joints with connectors.

The force $P/2$ times its distance from the center of the connector, o plane of the joint, induces an overturning moment which tends to caus the joint to spread apart. There are three things counteracting thi

tendency. First is the resistance of the wood in compression perpendicular to the grain which may be represented by Q. Second is the friction between the members and between the ring and the members. Finally, the bolthead, nut, and washers transmit tension to the bolt.

The pressure distribution of force P is shown as trapezoidal because the deformation of the wood causes a slight rotation when the load is applied. This slight rotation tends to shift the connector so that P tends to move toward the plane of the joint. If the grooves for the connector were cut considerably deeper than required, the stress distribution could even become triangular.

48. Fabrication. The split rings, shear plates, and claw plates are installed into grooves or daps that are cut by means of special tools.

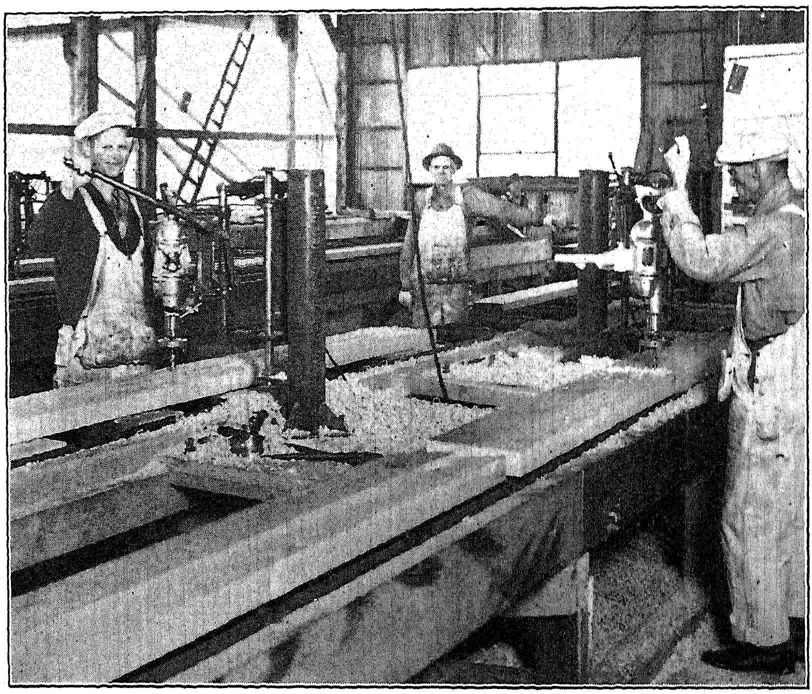


Fig. 27. Fabricating truss members in shop of Timber Structures, Inc., Portland, Oregon.

The Timber Engineering Company manufactures two types, one consisting of a circular cutter head with replaceable knives and a pilot and the other consisting of a two-wing cutter head with knives attached and a pilot.

Claw plates fit partly into daps, and the teeth are embedded into the wood by driving or by pressure. The toothed rings and spike grids are installed by pressure, the best procedure for which is to use a high-strength rod and ball-bearing washer.

The high-strength rod should have the same diameter as the bolt to be used in the joint and should be used with plate washers whose diameters equal the diameters of the connectors.

Flat clamping plates are embedded into the wood by pressure or driving. Usually a driving ram is used in combination with a heavy steel seating block with slots to protect the teeth. Flanged clamping plates have teeth on one side only and are driven into the guard timber by means of a heavy ram and a cover plate to protect the plate against bending.

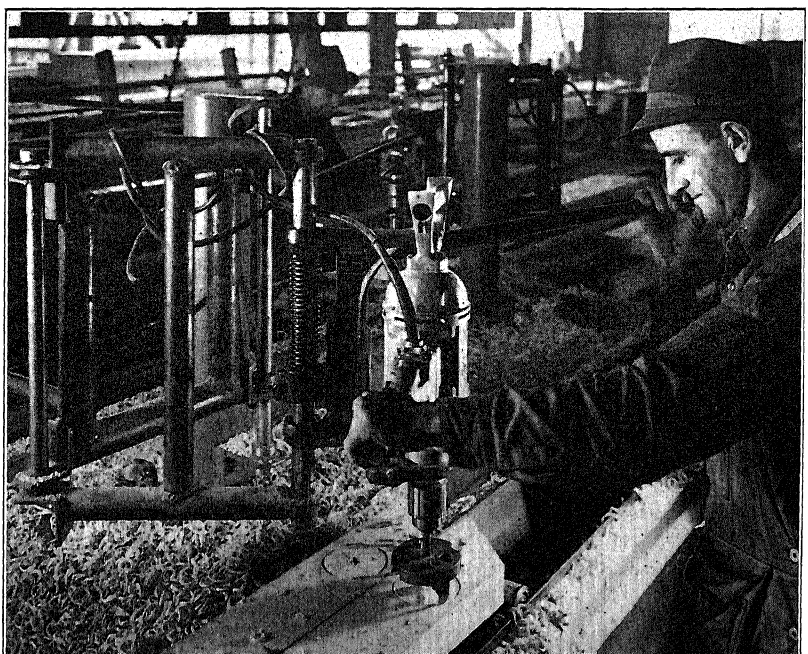


FIG. 28. Cutting grooves for split rings in shop of Timber Structures, Inc.

When several identical trusses or arches are to be used in a structure, it is best to prepare templates for the individual members. These should have the bolt holes in them and should be cut to the exact shape of the member. Additional markings can be used on the templates for locating grooves or for identification purposes.

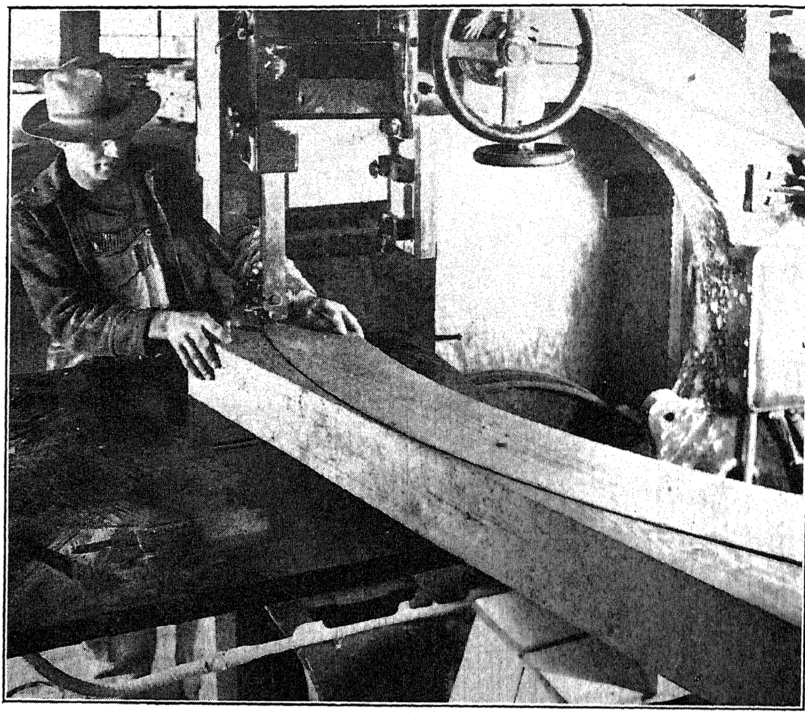


FIG. 29. Fabrication of curved chord members.

ILLUSTRATIVE PROBLEMS

Problem. Find the number and size of split ring connectors to carry 20,000 lb. in a lap tension joint. Use a dense No. 1 structural grade of southern pine (1400# f).

$$\text{Required area} = \frac{20{,}000}{1400} = 14.29 \text{ sq. in.}$$

A 3″ x 6″ has an area of 14.77 sq. in.
A 2″ x 10″ has an area of 15.44 sq. in.

Either two rows of 2½-in. rings or one row of 4-in. rings can be used in the 2″ x 10″ or one row of 4-in. rings can be used in the 3″ x 6″.

No. of Split Rings Required:

From Chart 9, the safe load for 2½-in. rings in one face of 1⅝-in. lumber $= 3300$ lb. No. of rings $= \dfrac{20{,}000}{3300} = 6.$

The safe load for 4-in. rings in one face of 1⅝-in. lumber = 6400 lb. No. of 4-in. rings $= \dfrac{20{,}000}{6400} = 3.125.$ Use 4. This same number is required in 2⅝-in. lumber.

It is best to keep the number of bolts at a minimum and the 4-in. rings with 3″ x 6″ material are recommended.

Net Section:

From Table 19, the constant 0.00041 is selected.

Required net section = 20,000 $\times$ 0.00041 = 8.20 sq. in.

From Table 20, the projected area of a 4-in. ring and bolt in lumber 2⅝ in. thick = 3.84 sq. in.

Actual net section = 14.77 − 3.84 = 10.93 sq. in.

Reduced Spacing:

Capacity of rings = 4 $\times$ 6400 = 25,600 lb.

Percentage of capacity developed $= \dfrac{20{,}000}{25{,}600} = 78$ per cent.

Total reduction in joint = 400 − 4 $\times$ 78 = 88 per cent.

Reduced spacing $= \dfrac{88}{3} = 29$ per cent.

Standard spacing − minimum spacing at reduction of 50 per cent = 9 − 4⅞ = 4⅛ in.

Reduced spacing at 29 per cent $= 9 - \dfrac{4.125 \times 0.29}{0.50} = 6\tfrac{5}{8}$ in.

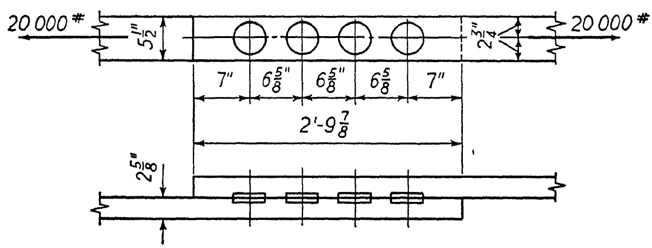


FIG. 30. Lap tension joint using split rings.

Problem. Design and detail a tension joint, using split ring connectors to carry 55,000 lb. Use dense No. 1 structural southern pine (1400# f).

Size of Members: $\dfrac{55{,}000}{1400} = 39.25$ sq. in.

Use 6″ x 8″, area = 41.25 sq. in.

Use 3″ x 8″ splice plates.

No. of 6-in. rings $= \dfrac{55{,}000}{10{,}500} = 5.3$. Use 6.

Spacing Parallel to Grain: Standard = 12 in. Minimum = 7 in.

End Distance: Standard = 9 in. Minimum = 4½ in.

Edge Distance: Standard and Minimum = 3¾ in.

Net Area: Required = 55,000 $\times$ 0.00051 = 28.0 sq. in.

Actual = 41.25 − 10.79 = 30.46 sq. in.

Reduced Spacing:

$$\text{Percentage of capacity developed} = \frac{55{,}000}{6 \times 10{,}500} = 88 \text{ per cent.}$$

Total reduction in joint $= 300 - 264 = 36$ per cent.

$$\text{Reduction for spacing} = \frac{36}{2} = 18 \text{ per cent.}$$

Standard spacing $-$ minimum spacing $= 12 - 7 = 5$ in.

$$\text{Reduced spacing} = 12 - \frac{5 \times 18}{50} = 10\tfrac{1}{4} \text{ in.}$$

Joint length $= 2(9 + 10\frac{1}{4} + 10\frac{1}{4} + 9) = 6$ ft. 5 in.

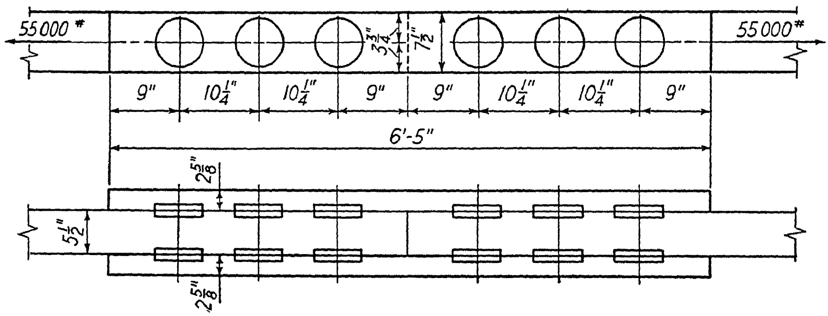


FIG. 31. Split ring tension joint.

Remarks: A comparison of this joint with the one designed in Article 39 for bolts only will show the advantages in using connectors.

Problem. Design a compression joint, using split ring connectors to carry 55,000 lb. Use dense select structural Douglas fir.

Size of Members: Assume main member to be a short column with an L/D ratio less than 11.

$$\text{Required area} = \frac{55{,}000}{1300} = 42.3 \text{ sq. in. Use 6 by 10 in., area} = 52.25 \text{ sq. in.}$$

$$\text{No. of 6-in. rings} = \frac{55{,}000}{10{,}500} = 5.6. \text{ Use 6.}$$

Spacing Parallel to Grain: Standard = 12 in. Minimum = 7 in.
Edge Distance: Standard and Minimum = $3\frac{3}{4}$ in.
End Distance: Standard = $7\frac{1}{2}$ in. Minimum = $4\frac{1}{4}$ in.
Net Area:

Required $= 55{,}000 \times 0.00051 = 28.0$ sq. in.
Actual $= 52.25 - 10.79 = 41.46$ sq. in.

Reduced Spacing:

Total reduction in joint = 36 per cent.
Reduced spacing = $10\frac{1}{4}$ in.
Joint length $= 2(7\frac{1}{2} + 10\frac{1}{4} + 10\frac{1}{4} + 7\frac{1}{2}) = 5$ ft. 11 in.

Remarks: This joint is very similar to the one in the previous example, but illustrates the fact that the length of a compression joint is usually less than a tension joint designed for the same load because the required end distance is less.

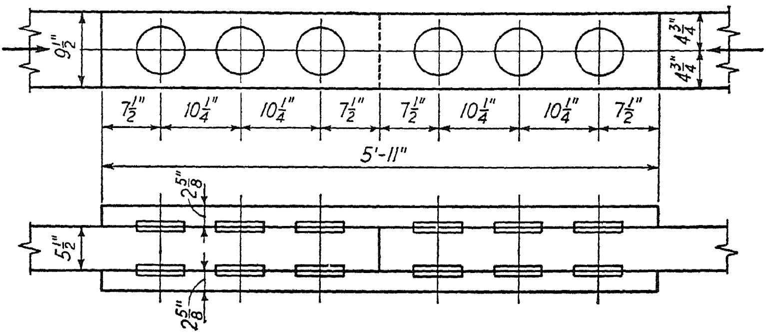


Fig. 32. Split ring compression joint.

If this joint was supported in a lateral direction, it would be possible to use a metal bearing plate between the ends of the members and develop between 50 per cent and 75 per cent of the bearing stress of the members. In this case the number of 6-in. rings required would become $\frac{55{,}000 \times 0.25}{10{,}500} = 1.3$. Use two 6-in. rings on each side of the joint.

49. Multiple Joints. Because the load-carrying capacity of connectors decreases as the angle between the direction of the applied load and the grain of the wood increases, it is important to know which member in a joint is being acted upon at an angle to the grain. In the heel joint of a truss the vertical component of the load transmitted by the top chord is usually counteracted by the reaction of the supporting column of the truss. This leaves the horizontal component, which is the stress in the bottom chord; and the load on the connectors in this member is parallel to the grain. However, an equal and opposite load which acts on the connectors in the top chord is acting at an angle to the grain in this member equal to the angle between the two members. The number of connectors in the joint then is determined by their load capacities at this angle.

For a joint where two web or diagonal members enter on each side of a horizontal member, the vertical reactions are usually equal and opposite in direction, and the joint is designed with load capacities of the connectors determined by the angle of load to grain that each side member makes with the center member. In a three-member joint it is generally the center member in which the load acts at an angle to the grain; in a five-member joint it is usually the second and fourth members that are loaded at an angle to the grain.

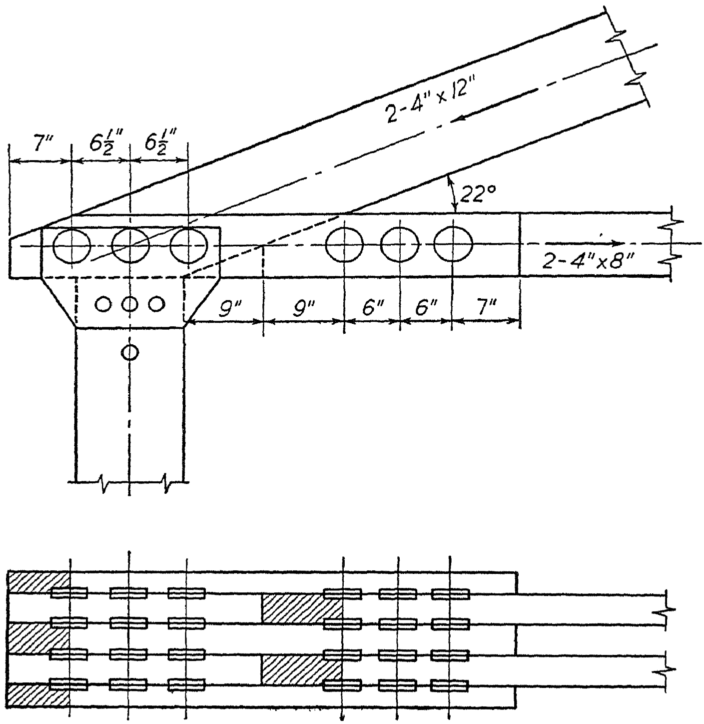


Fig. 33. Typical heel joint.

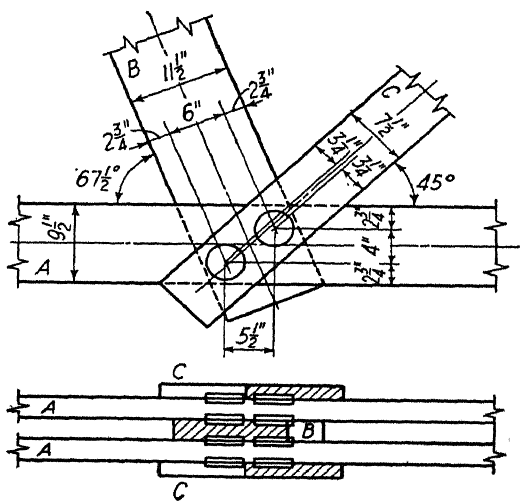


Fig. 34. Typical chord joint in a Fink truss.

When two or more members form a joint and the angle of load to grain is less than 30° for any two contacting members, the connectors can be spaced with reference to any member. This situation occurs in the peak joint of a Fink truss.

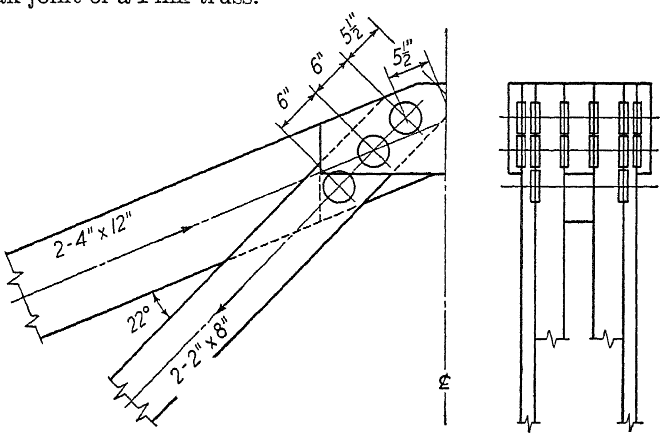


FIG. 35. Typical peak joint in a Fink truss.

When the members enter a joint such as is found in a flat-top Pratt truss, and the angle between the members is greater than 30°, the spacing of the connectors must be considered in the member that has the load applied at the greatest angle.

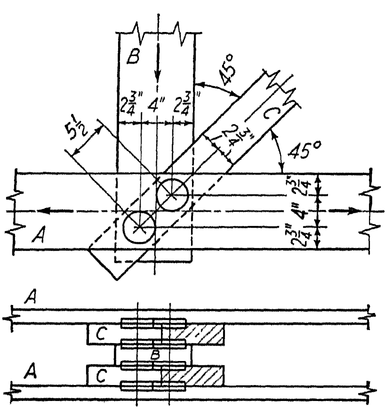


FIG. 36. Typical joint in a Pratt truss.

DESIGN EXAMPLE. Consider the joints shown in Figures 37 and 38, and determine the size of the members and the number of connectors to satisfy the conditions of the joint. As shown in the figures, there are two methods of placing the connectors in the joint. They can be spaced either parallel or perpendicular to the grain in member No. 2.

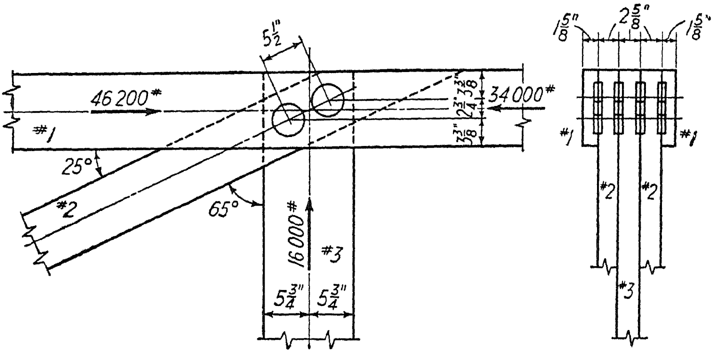


FIG. 37. Five-member split ring joint.

Split-Ring Connector Data	Vertical to Diagonals	Chords to Diagonals
Size	4″	4″
Number of faces	2	2
Thickness of piece	2⅝″	2⅝″
Angle	65°	25°
Standard design load	4,950 lb.	5,790 lb.
Joint Stress	16,000 lb.	12,200 lb.
Connectors Required	$\frac{16,000}{4,950} = 3.24$	$\frac{12,200}{5,790} = 2.11$
Connectors Used	4	4
Capacity Developed	$\frac{3.24}{4} = 81\%$	$\frac{2.11}{4} = 52.8\%$
Spacing Required	5½″	9″ to 5½″
Spacing Used	5½″	5½″
Edge Distance:		
Comp. Side Diagonal	2¾″	2¾″
Vertical	2¾″	2¾″
Chord	2¾″	2¾″
Width of Diagonal	2¾ + 2¾ = 5½″	
Width of Vertical	2¾ + 5 + 2¾ = 10½″	
Width of Chord	2¾ + 2.75 + 2¾ = 8.25″	
Size of Members:		
No. 1 Chords	2—2″ x 10″	
No. 2 Diagonals	2—3″ x 6″	
No. 3 Vertical	1—3″ x 12″	

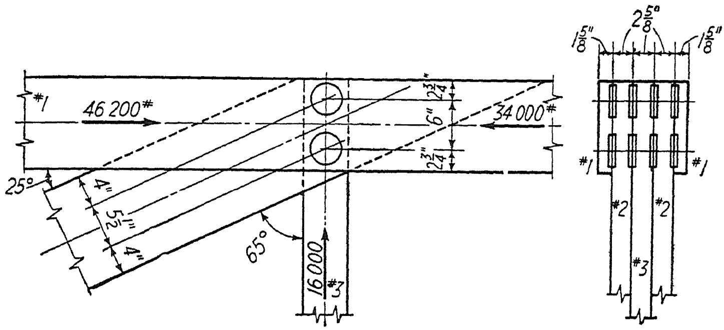


FIG. 38. Five-member split ring joint.

Split-Ring Connector Data	Vertical to Diagonals	Chords to Diagonals
Size	4″	4″
Number of faces	2	2
Thickness of piece	2⅝″	2⅝″
Angle	65°	25°
Standard design load	4,950 lb.	5,790 lb.
Joint Stress	16,000 lb.	12,200 lb.
Connectors Required	$\frac{16{,}000}{4{,}950} = 3.24$	$\frac{12{,}200}{5{,}790} = 2.11$
Connectors Used	4	4
Capacity Developed	$\frac{3.24}{4} = 81\%$	$\frac{2.11}{4} = 52.8\%$
Spacing Required	6½″ to 5½″	5½″
Spacing Used	5½″	5½″
Edge Distance:		
Comp. Side Diagonal	3½″	3½″
Chord	2¾″	2¾″
Vertical	2¾″	2¾″
Width of Diagonal	3½ + 5½ + 3½ = 12½″	
Width of Chord	2¾ + 6 + 2¾ = 11½″	
Width of Vertical	2¾ + 2¾ = 5½″	
Size of Members:		
No. 1 Chords	2—2″ x 12″	
No. 2 Diagonals	2—3″ x 14″	
No. 3 Vertical	1—3″ x 6″	

PROBLEMS

1. Find the safe working stress when a load is applied at an angle of 60° in a dense No. 1 structural grade of southern pine.

2. Calculate the safe working stress for a select structural grade of Douglas fir (coast region) when the load is applied at an angle of 45° to the grain.

3. Determine the number and size of nails and common screws to resist a pull of 3000 lb. in the direction of their length when they are used to fasten a piece 2 in. thick to a piece 6 in. thick. Use Douglas fir.

4. Find the safe load for a tension joint with wood splice plates and six ½-in. bolts on each side of the joint. Use (*a*) southern pine; (*b*) eastern hemlock; (*c*) tupelo. Assume a main member 4 in. thick.

5. Find the safe load for a bolted joint with six ½-in. bolts when the load is applied at an angle of 45°. Assume a main member 4 in. thick and two side members each 2 in. thick. Use (*a*) Douglas fir (Rocky Mountain region); (*b*) rock elm; (*c*) red oak.

6. Find the number and size of spikes, lag screws, and bolts to resist a lateral force of 3000 lb. between a 2-in. and 4-in. piece of seasoned southern pine. (*a*) Consider the load parallel to the grain of both pieces. (*b*) Consider the load perpendicular to the grain of the 4-in. piece.

7. Design and detail a bolted joint to carry 20,000 lb. in tension. Assume that the joint will be exposed to the weather. Use southern pine and consider the main member 6 in. thick. (*a*) Load applied parallel to grain in main member. (*b*) Load applied perpendicular to grain in main member.

8. Find the number and size of bolts and split ring connectors with bolts in each member of a two-member joint when a load of 15,000 lb. is applied in tension in one of the members. The members butt at an angle of 45°. Use dense select Douglas fir and consider both members to be 4 in. thick.

9. Two members 6 in. thick join at an angle of 60° and one member carries a load of 22,000 lb. in compression. Using metal splice plates and bolts, design and detail the joint. Use southern cypress.

10. In a lap tension joint carrying 30,000 lb. determine the size of the members, the number of split rings required, and detail the joint, taking full advantage of reduction in spacing and end distance. Use dense No. 1 structural southern pine.

CHAPTER IV

BEAMS AND COLUMNS

BEAMS

50. General. In designing wood beams three stresses must be considered. These are the extreme fiber stress in flexure, the maximum horizontal shear stress, and the stress in compression across the grain at the reactions or concentrated loads. Besides these stresses, which must not exceed the allowable value in each case, the beam must possess the stiffness required in the particular use for which it is intended.

51. Extreme Fiber Stress. The size of a beam to resist a bending moment due to a load may be determined by the familiar formula $M = \frac{fI}{c}$ in which M is the bending moment, f is the allowable fiber stress for the grade of lumber used, I is the moment of inertia of the section, and c is the distance from the neutral axis to the extreme fiber. For practical purposes it is assumed that the neutral axis is located at the center of the beam; however, this is not exactly correct because wood is a nonhomogeneous material.

A wood beam will carry a much greater load for a few minutes than for several years, and for this reason impact stresses that do not exceed the live load stresses may be neglected.

52. Horizontal Shear. When the design of rectangular beams is governed by the allowable unit stress in horizontal shear, the usual formula is $q = \frac{3}{2}\frac{V}{bd}$, in which q represents the maximum horizontal shear stress, V is the external shear, b is the width of the beam, and d is the depth. However, this formula should be modified because the grading rules for structural lumber permit certain depths of checks and shakes at the neutral plane; tests at the Forest Products Laboratory showed that when these checks or shakes are present, the upper and lower portions of a beam act partly as two beams and partly as a unit. For this reason part of the end reaction is resisted internally by each half of the beam acting

independently and consequently is not associated with shearing stress at the neutral plane.[1]

The following procedure is recommended by the Laboratory for calculating the horizontal shear on the neutral plane in checked beams:

1. Use the ordinary shear formula and the unit stress in horizontal shear for the particular structural grade selected.

2. In calculating the reaction for use in the formula: (*a*) disregard all loads within the height of the beam from both supports; (*b*) place the heaviest moving load at three times the height of the beam from the support; and (*c*) treat all other loads in the usual manner.

If the beam does not qualify under the foregoing recommendation, the reactions should be determined by the following precise equation:

$$R = \frac{10P(L - x)\left(\frac{x}{h}\right)^2}{9L\left[2 + \left(\frac{x}{h}\right)^2\right]}$$

R is the reaction due to the load P, L the span in inches, x the distance in inches from the reaction to the load P, and h the height of the beam in inches.

If the load is a moving one, the value of x for use in the above formula may be found from the following equation:

$$Z^3 + 6Z = 4U$$

where $Z = x/h$ and $U = L/h$.

53. Bearing. The allowable unit stress in compression perpendicular to the grain should be equal to or greater than the value obtained by dividing the end reaction by the end-bearing area of the beam. The working stresses in compression perpendicular to the grain given in Table 3 apply to bearings of any length at the ends of a beam and to bearings of 6 in. or more at any other point in the length of the beam. If the bearings are shorter than 6 in. and located 3 in. or more from the end of a beam the working stresses may be increased in accordance with the values given in Table 5.

54. Deflection. The dimensions of a beam designed for stiffness may be calculated by means of the usual deflection formulas, and the deflection is generally limited to 1/360 of the span. For highway bridges a ratio of 1/200 of the span is commonly used and for stringers in railroad bridges and trestles limitations range from 1/200 to 1/300. Wood

[1] "New Method of Calculating Longitudinal Shear in Wooden Beams," by Newlin, Heck, and March, *Eng. News-Record*, 110: 594–596.

beams usually sag in time, which means that the deflection will increase. For this reason it is customary to double the dead load in computing the deflection.

55. Notched Beams. Often beams are notched at the ends to allow more clearance or to bring the top surfaces level with adjacent beams. Notches at intermediate points are not uncommon because it is frequently necessary to allow space for pipes and other parts of a structure.

When beams are notched at the end as shown in Figure 39, the bending load must be checked against the load obtained by the following formula:

$$V = \frac{2}{3}\frac{bd_1^2q}{d}$$

V = end reaction.
b = width of beam.
d_1 = end depth above the notch.
q = working stress in horizontal shear.
d = total depth of beam.

When notches are located at or near the middle of a beam, the net depth should be used in obtaining the bending strength.

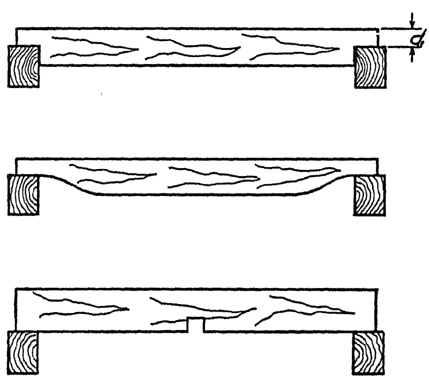


Fig. 39. Notched beams.

56. Buckling. Narrow and deep beams unsupported laterally will often fail in torsion due to lateral buckling. The buckling load for a beam supported and held vertical at its ends, but not otherwise restrained, and carrying a uniformly distributed load may be calculated from the following formula:

$$W = \frac{28.3\sqrt{EI^1GK}}{L^2}$$

For a beam under the same conditions as above except that, in addition to being held vertical, it is clamped at the ends to prevent lateral rotation, the formula becomes

$$W = \frac{43.3\sqrt{EI^1GK}}{L^2}$$

W = buckling load.
E = modulus of elasticity.
I^1 = moment of inertia of beam about its vertical axis.
G = modulus of rigidity in torsion = $E/16$.
L = span, in inches.
K = torsion constant of the section and is equal to βdb^3 in which β is a constant depending upon the ratio of the sides d/b.

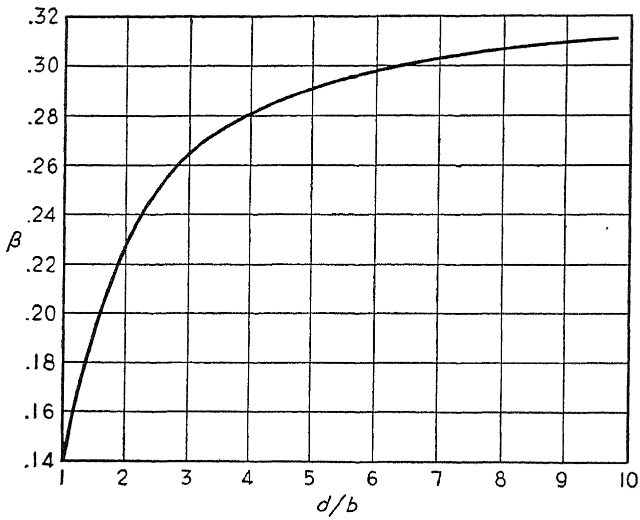


CHART 12. Values of β in terms of the ratio of the sides d/b.

ILLUSTRATIVE PROBLEMS

Problem. Design a beam with a span of 16 ft. 0 in. to carry a load of 1000 lb. per linear ft. The beam is to be of Douglas fir, for which the allowable unit fiber stress is 1600 lb. per sq. in. and the allowable unit horizontal shear is 100 lb. per sq. in. The modulus of elasticity is 1,600,000 lb. per sq. in.

$$M = \frac{wl^2}{8} = \frac{1000 \times 16 \times 16 \times 12}{8} = 384{,}000 \text{ in.-lb.}$$

$$V = \frac{wl}{2} = \frac{1000 \times 16}{2} = 8000 \text{ lb.}$$

$$f = \frac{Mc}{I} = \frac{M6}{bd^2} \qquad bd^2 = \frac{384{,}000 \times 6}{1600} = 1438 \text{ in.}^3$$

Let $b = 9.5$ in. (10 in. nominal)

$$d = \sqrt{\frac{1438}{9.5}} = 12.6 \text{ in.}$$

A beam 10 by 14 in. will satisfy the bending requirements.

$$V^1 = \frac{1000(16 - 2.33)}{2} = 6888 \text{ lb.}$$

$$q = \frac{3}{2}\frac{V^1}{bd} \qquad bd = \frac{3 \times 6888}{2 \times 100} = 103.32 \text{ sq. in.}$$

The area of a 10 by 14 in. member $= 9.5 \times 13.5 = 128$ sq. in., which furnishes ample shearing resistance.

$$\text{Deflection, } y = \frac{5}{384} \times \frac{wl^4}{EI} \qquad I \text{ for a 10 by 14 in. member} = 1947.8$$

$$y = \frac{5 \times 16{,}000(16 \times 12)^3}{384 \times 1{,}600{,}000 \times 1947.8} = 0.471 \text{ in.}$$

$$\text{Allowable deflection} = \frac{1}{360} \text{ of the span.}$$

$$y = \frac{16 \times 12}{360} = 0.534 \text{ in.}$$

Problem. Design a built-up beam using 6 by 8 in. timbers and 4-in. split rings to carry 700 lb. per ft. on a 20-ft. span. Use dense No. 1 structural southern pine (1400 f).

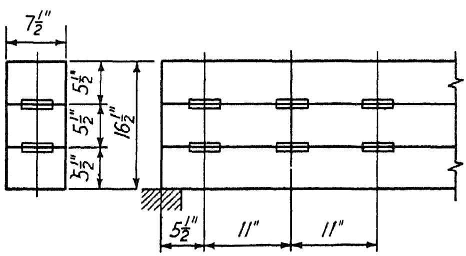


FIG. 40. Built-up beam with split rings.

Assume weight of beam = 35 lb. per ft.

$$f = \frac{M}{s} \quad s = \frac{735 \times 20 \times 20 \times 12}{8 \times 1400} = 315 = \frac{bd^2}{6} \quad d^2 = \frac{315 \times 6}{7.5} \quad d = 16 \text{ in.}$$

Use 3, 6 by 8-in. timbers. $V = (700 + 35)10 = 7350$ lb.

$$q = \frac{3 \times 7350}{2 \times 7.5 \times 16.5} = 89 \text{ lb. per sq. in.}$$

Allowable = 100 lb. per sq. in.

$$\text{Shear at plane of contact} = \frac{VQ}{Ib} = \frac{7350 \times 7.5 \times 5.5 \times 5.5}{\frac{7.5 \times 16.5^3 \times 7.5}{12}} = 78 \text{ lb. per sq. in.}$$

Shear per linear foot $= 78 \times 12 \times 7.5 = 7025$ lb.

Spacing, 4-in. split rings $= \dfrac{0.5 \times 6400 \times 2}{7025} \times 12 = 11$ in.

At ¼ pt. $V = 3675$ lb.
Spacing $= 22$ in.

End bearing $\dfrac{7350}{380} = 19.4$ sq. in. Length $= \dfrac{19.4}{7.5} = 2\frac{3}{4}$ in.

Problem. Design the floor system illustrated in Figure 41.

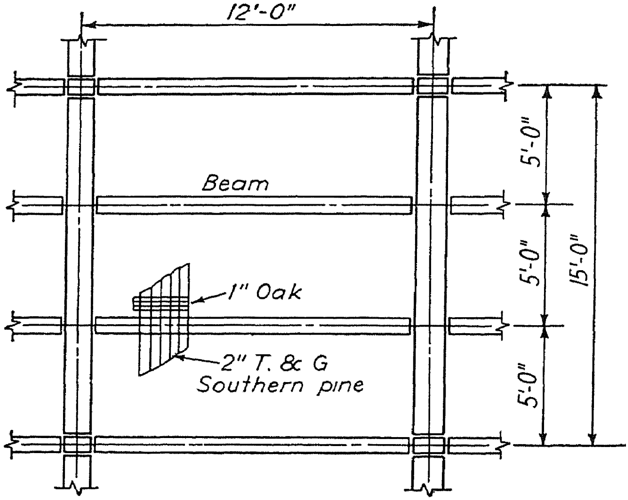


FIG. 41. Floor system.

Loads: Live load, 80 lb. per sq. ft.
Weight of wood, 40 lb. per cu. ft.

Materials:
Beams and girders, dense No. 1 structural southern pine.
Sub-flooring, 2-in. T. & G. southern pine.
Flooring, 1-in. T. & G. oak.

Working stresses, southern pine beams and girders:
Flexure, 1400 lb. per sq. in.
Compression parallel to grain, 1000 lb. per sq. in.
Compression perpendicular to grain, 380 lb. per sq. in.
Horizontal shear, 100 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Southern pine sub-flooring:
Flexure, 1000 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Oak flooring:

Flexure, 1000 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Flooring loads:

Live load = 80 lb. per sq. ft.

$$\text{Dead load} = \frac{1.625 \times 40}{12} + \frac{0.75 \times 40}{12} = 7.92 \text{ lb. per sq. ft.}$$

Total load = 80 + 7.92 = 87.92 lb. per sq. ft.

Flooring flexure:

$$\text{Maximum moment} = \frac{87.92 \times 5 \times 5 \times 12}{10} = 2640 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{2640}{1000} = 2.64 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{12 \times 1.625^2}{6} = 5.28 \text{ in.}^3$$

Flooring deflection:

The dead load is increased by 100%, and the total load = $(7.92 \times 2 + 80)5 = 479$ lb. per sq. ft.

$$\text{Maximum deflection} = \frac{3}{640} \cdot \frac{479 \times 60^3}{1{,}600{,}000 \times \frac{1}{12} \times 12 \times 1.625^3} = 0.071 \text{ in.}$$

$$\text{Allowable deflection} = \frac{60}{360} = 0.167 \text{ in.}$$

Beam loads:

Live load per linear foot = $80 \times 5 = 400$ lb.
Load due to flooring = $7.92 \times 5 = 39.6$ lb.

$$\text{Assuming an 8 by 10 in. beam, weight} = \frac{7.5 \times 9.5 \times 40}{12 \times 12} = 19.7 \text{ lb.}$$

Total load = 400 + 39.6 + 19.7 = 459.3 lb. per ft.

Beam flexure:

$$\text{Maximum moment} = \frac{459.3 \times 12 \times 12 \times 12}{8} = 99{,}200 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{99{,}200}{1400} = 70.8 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{7.5 \times 9.5^2}{6} = 112.81 \text{ in.}^3$$

Beam shear:

$$q = \frac{3}{2}\frac{V}{bd} = \frac{3 \times 459.3 \times 6}{2 \times 7.5 \times 9.5} = 58 \text{ lb. per sq. in.}$$

Allowable shear = 100 lb. per sq. in.

Beam deflection:

The dead load is increased by 100%, and the total load = $(59.3 \times 2 + 400)12 = 6230$ lb.

$$\text{Maximum deflection} = \frac{5}{384} \cdot \frac{6230 \times 144^3}{1{,}600{,}000 \times 535.8} = 0.283 \text{ in.}$$

$$\text{Allowable deflection} = \frac{12 \times 12}{360} = 0.40 \text{ in.}$$

Girder loads:

Concentrated loads $= 459.3 \times 12 = 5520$ lb.

$$\text{Assuming a 12 by 14 in. girder, weight} = \frac{11.5 \times 13.5 \times 40}{12 \times 12} = 43.1 \text{ lb. per ft.}$$

Girder flexure:

$$\text{Maximum moment} = 5520 \times 60 + \frac{43.1 \times 15 \times 15 \times 12}{8} = 355{,}500 \text{ in.-lb.}$$

$$\text{Required section modulus} = \frac{355{,}500}{1400} = 252 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{11.5 \times 13.5^2}{6} = 349.3 \text{ in.}^3$$

Girder shear:

$$V = \frac{43.1 \times 15}{2} + 5520 = 5843 \text{ lb.}$$

$$q = \frac{3 \times 5843}{2 \times 11.5 \times 13.5} = 56.5 \text{ lb. per sq. in.}$$

Allowable shear = 100 lb. per sq. in.

Girder deflection:

Weight of girder increased 100% $= 43.1 \times 2 = 86.2$ lb. per ft.
Maximum deflection due to weight of girder =

$$\frac{5 \times 86.2 \times 15 \times 15^3 \times 12^3}{384 \times 1{,}600{,}000 \times 2358} = 0.026 \text{ in.}$$

Weight of beam and flooring increased 100% $= 59.3 \times 2 \times 12 = 1420$ lb.
Live load $= 400 \times 12 = 4800$ lb.
Total concentrated load = 6220 lb.
Maximum deflection due to concentrated loads =

$$\frac{23 \times 6220 \times 15^3 \times 12^3}{648 \times 1{,}600{,}000 \times 2358} = 0.342 \text{ in.}$$

Total deflection $= 0.026 + 0.342 = 0.368$ in.

$$\text{Allowable deflection} = \frac{15 \times 12}{360} = 0.50 \text{ in.}$$

57. Beams with Inclined Loads. As the direction of the load acting on rafters and purlins is usually not perpendicular to one of the principal axes, it is necessary to resolve the load into components parallel and perpendicular to the vertical axis of the member.

EXAMPLE: Design a rafter fastened to purlins and inclined at an angle of 30° as shown in Figure 42. Span = 12 ft. 0 in.

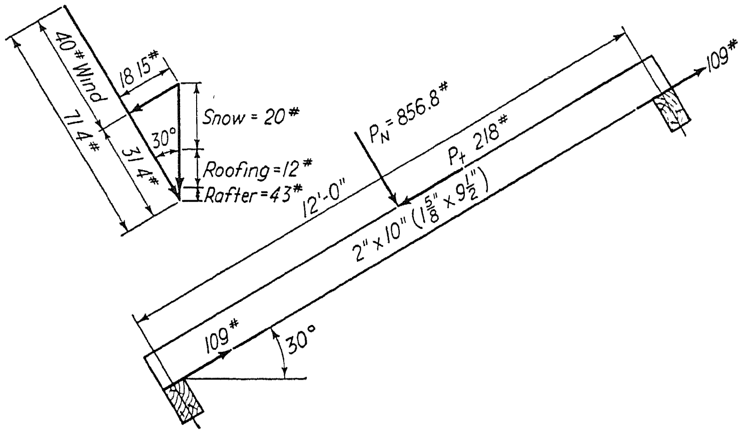


FIG. 42. Loads acting on inclined rafter.

Vertical loads:

Snow, 20 lb. per ft.
Roofing, 12 lb. per ft.
Rafter, 40 lb. per cu. ft.

Normal loads:

Wind, 40 lb. per ft.

Working stresses:

Flexure, 900 lb. per sq. in.
Horizontal shear, 100 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Flexure:

Assume a 2 by 10-in. member.

$$\text{Weight of rafters} = \frac{1.625 \times 9.5 \times 40}{12 \times 12} = 4.3 \text{ lb. per ft.}$$

Total vertical load = $20 + 12 + 4.3 = 36.3$ lb. per ft.
Total normal load = $36.3 \times 0.866 + 40 = 71.4$ lb. per ft.

$$\text{Moment due to normal load} = \frac{71.4 \times 12 \times 12 \times 12}{8} = 15{,}400 \text{ in.-lb.}$$

Moment is also caused by the eccentricity of the tangential force. This force $= 36.3 \times 0.5 = 18.15$ lb. per ft.

Total tangential force $= 18.15 \times 12 = 218$ lb.

Assuming one-half of this force to be resisted at each purlin, the moment becomes $\frac{218}{2} \times 9.5 = 1030$ in.-lb.

Total moment $= 15{,}400 + 1030 = 16{,}430$ in.-lb.

$$\text{Required section modulus} = \frac{16{,}430}{900} = 18.3 \text{ in.}^3$$

$$\text{Section modulus provided} = \frac{1.625 \times 9.5^2}{6} = 24.44 \text{ in.}^3$$

Shear:

$$V = \frac{71.4 \times 12}{2} = 428 \text{ lb.}$$

$$q = \frac{3 \times 428}{2 \times 1.625 \times 9.5} = 41.7 \text{ lb. per sq. in.}$$

Deflection:

Normal dead load $= 16.3 \times 0.866 = 14.1 \times 2 = 28.2$ lb. per ft.

Normal snow load $= 20 \times 0.866 = 17.3$ lb. per ft.

Normal wind load $= 40$ lb. per ft.

Total load $= (28.2 + 17.3 + 40)12 = 1026$ lb.

$$\text{Maximum deflection} = \frac{5 \times 1026 \times 144^3}{384 \times 1{,}600{,}000 \times 116} = 0.215 \text{ in.}$$

$$\text{Allowable deflection} = \frac{12 \times 12}{360} = 0.4 \text{ in.}$$

Example. Design a purlin to span 16 ft. 0 in. with the roof inclined at 30°. Spacing of purlins = 5 ft. 0 in.

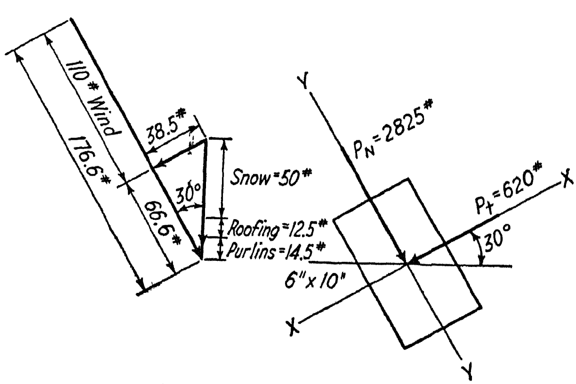


Fig. 43. Inclined purlin.

Vertical loads:

Snow, 10 lb. per sq. ft.
Sheathing, 2.5 lb. per sq. ft.
Weight of purlin = 40 lb. per cu. ft.

Normal loads:

Wind, 22 lb. per sq. ft.

Working stresses:

Flexure, 1400 lb. per sq. in.
Horizontal shear, 100 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Flexure:

Assume a 6 by 10 in. purlin.

$$\text{Total vertical load} = 12.5 \times 5 + \frac{5.5 \times 9.5}{12 \times 12} \times 40 = 77 \text{ lb. per ft.}$$

$$\text{Total normal load} = 77 \times 0.866 + 22 \times 5 = 176.6 \text{ lb. per ft.}$$

$$\text{Maximum moment} = \frac{176.6 \times 16 \times 16 \times 12}{8} = 68{,}000 \text{ in.-lb.}$$

$$\text{Section modulus provided} = \frac{5.5 \times 9.5^2}{6} = 84.6 \text{ in.}^3$$

$$\text{Fiber stress due to normal load} = \frac{68{,}000}{84.6} = 803 \text{ lb. per sq. in.}$$

$$\text{Tangential component} = 77 \times 0.5 = 38.5 \text{ lb. per ft.}$$

$$\text{Maximum moment} = \frac{38.5 \times 16 \times 16 \times 12}{8} = 14{,}800 \text{ in.-lb.}$$

$$\text{Section modulus provided} = \frac{9.5 \times 5.5^2}{6} = 47.8 \text{ in.}^3$$

$$\text{Fiber stress due to tangential component} = \frac{14{,}800}{47.8} = 310 \text{ lb. per sq. in.}$$

$$\text{Total fiber stress required} = 803 + 310 = 1113 \text{ lb. per sq. in.}$$

Shear:

$$V = \frac{176.6 \times 16}{2} = 1410 \text{ lb.}$$

$$q = \frac{3 \times 1410}{2 \times 5.5 \times 9.5} = 40.6 \text{ lb. per sq. in.}$$

Deflection:

$$\text{Normal dead load} = (2.5 \times 5 + 14.5)0.866 \times 2 = 46.7 \text{ lb. per ft.}$$
$$\text{Normal snow load} = 10 \times 5 \times 0.866 = 43.3 \text{ lb. per ft.}$$
$$\text{Normal wind load} = 22 \times 5 = 110 \text{ lb. per ft.}$$
$$\text{Total normal load} = (46.7 + 43.3 + 110)16 = 3200 \text{ lb.}$$

$$\text{Maximum deflection} = \frac{5 \times 3200 \times 16^3 \times 12^3}{384 \times 1{,}600{,}000 \times 402} = 0.458 \text{ in.}$$

$$\text{Allowable deflection} = \frac{16 \times 12}{360} = 0.533 \text{ in.}$$

COLUMNS

58. Classification. Wood columns are divided into three general classes depending upon the ratio of the unsupported length of the column to its least dimension. For short columns in which the unsupported length divided by the least dimension does not exceed eleven, the full working value in compression may be used, and the formula used in design is $P/A = c$. P/A represents the load divided by the cross-sectional area, and c is the safe unit compressive stress parallel to the grain.

59. Long Columns. The general equation recommended for long columns of uniform cross section is the Euler formula in which

$$\frac{P}{A} = u\frac{\pi^2 E}{\left(\frac{L}{r}\right)^2}$$

P = maximum load on the column.
A = cross-sectional area.
E = modulus of elasticity.
L = length, in inches.
r = radius of gyration.
u = constant depending on end conditions.

r may be represented by $\frac{d}{\sqrt{12}}$, where d is the least dimension of the column. For pin-ended columns $u = 1$, and for columns with fixed ends $u = 4$.

Substituting 1 for u, and $\frac{d}{\sqrt{12}}$ for r in the above equation and introducing a factor of safety of 3, the formula becomes

$$\frac{P}{A} = \frac{0.274E}{\left(\frac{L}{d}\right)^2}$$

60. Intermediate Columns. For columns of the intermediate class, the Forest Products Laboratory has developed a fourth power parabolic formula as follows:

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{Kd}\right)^4\right]$$

The curve representing this formula passes through point c on the P/A axis and is tangent to the Euler curve at a point where P/A is two-

thirds of c. The value of c is the unit compressive stress for short columns, and the quantity K is a constant depending upon the grade and species of lumber used. K is the value of L/d at which the parabola is tangent to the Euler curve and may be represented by the formula

$$K = \frac{\pi}{2}\sqrt{\frac{E}{6c}} = 0.64\sqrt{\frac{E}{c}}$$

An intermediate column then is one whose L/d ratio is between eleven and K.

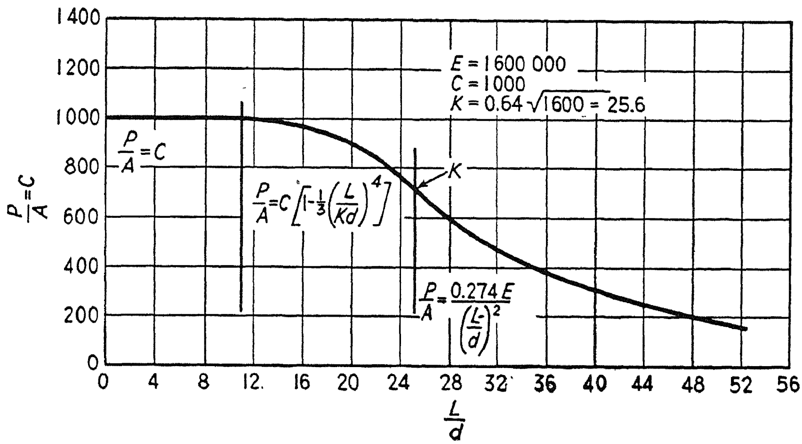


CHART 13. Graphical representation of column formulas.

61. Circular Columns. To compute the required size of a round column, design first for a square column and then use a round column having the same cross-sectional area as the square column. In calculating the equivalent area of a long tapered column, the cross-sectional area should be taken at a point one-third of its length from the bottom of the column.

62. Built-Up Columns. Many instances arise in the design of timber structures where it is economical to build up columns of considerable size from smaller pieces by spiking or bolting them together. The following table shows the strength of built-up columns in per cent of solid columns of similar grades. These percentages were developed by the Forest Products Laboratory and should be applied to the working stresses for solid columns of equal height, size, and grade. The percentages apply when the width of planks is not more than five times the thickness and when the spikes penetrate three planks. The longitudinal spacing of the spikes should be about six times the thickness of

Table 21. **Percentage of Solid Column Strength for Built-Up Columns**

[Wood Handbook, U. S. Dept. Agr., 1935]

L/d Ratios	*Percentages of Solid-Column Strength*	*L/d Ratios*	*Percentages of Solid-Column Strength*
6	82	18	65
10	77	22	74
14	71	26	82

one plank. When the L/d ratio is 10 or greater, the same percentages may be used for columns with butt-joined pieces.

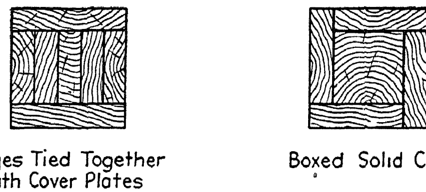


Fig. 44. Built-up columns.

63. Spaced Columns. The use of spaced columns joined with timber connectors is of importance in the design of compression members in

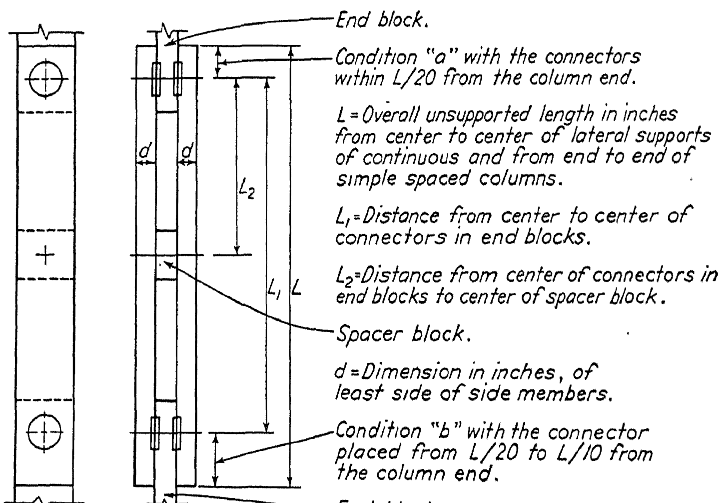


Fig. 45. Spaced column.

trusses and other structures. Spaced columns are formed of two or more members separated at their ends and middle point by blocking and joined at the ends by timber connectors.

In applying the formulas for solid columns to spaced columns it is necessary to introduce a restraint factor depending upon the distance from the center of the connector to the end of the piece.

LONG:

$$\frac{P}{A} = \frac{0.274\mathrm{E} \times 2.5}{(l/d)^2} \text{ for condition ``}a\text{''}$$

$$\frac{P}{A} = \frac{0.274\mathrm{E} \times 3}{(l/d)^2} \text{ for condition ``}b\text{''}$$

INTERMEDIATE:

For condition "a"

$$K_2 = 0.64\sqrt{\frac{2.5E}{c}} = K\sqrt{2.5} = K \times 1.5811$$

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{l}{K_2 d}\right)^4\right]$$

For condition "b"

$$K_3 = 0.64\sqrt{\frac{3E}{c}} = K\sqrt{3} = K \times 1.7320$$

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{l}{K_3 d}\right)^4\right]$$

The above formulas are based on the use of a spacer block within the middle 10 per cent of the length of the column and of end blocks so placed that they provide the proper end and edge distances for the connectors. Spacer and end blocks should be as thick as the individual column member. For compression members of trusses, the panel point may be considered as the end of the column and the web members as spacer blocks, providing the panel point is supported laterally.

The connectors at each end of a spaced column between the contacting face of the end block and an individual member of the column should be of a size and number to provide a load capacity in pounds equal to the product of the cross-sectional area of one of the members and the appropriate end-spacer block constant in Table 22.

Table 22. Constants for Use in Determining the Load Capacity of Connectors in Spaced Columns

[Wood Structural Design Data, Supplement 4, National Lumber Manufacturers' Association, 1939]

L/d Ratio of Individual Member in the Spaced Column	End-Spacer Block Constant		
	Group A Connector Loads	*Group B Connector Loads*	*Group C Connector Loads*
0 to 11	0	0	0
15	35	31	29
20	79	69	66
25	122	107	103
30	166	145	140
35	210	184	177
40	253	222	214
45	297	260	250
50	340	298	287
55	384	337	324
60 to 80	428	375	361

ILLUSTRATIVE PROBLEM

Problem. Design a column 12 ft. 9 in. high to carry a load of 100,000 lb. Use close-grained Douglas fir with a working stress of 1200 lb. per sq. in.

$$\text{Constant } K = 0.64\sqrt{\frac{E}{c}} = 0.64\sqrt{\frac{1{,}600{,}000}{1{,}200}} = 23.3$$

Assume a 10 by 10 in. column

$$\frac{L}{d} = \frac{12.75 \times 12}{9.5} = 16.1$$

Since L/d is greater than eleven but less than K, the column is in the intermediate class.

$$\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{Kd}\right)^4\right] = 1200\left[1 - \frac{1}{3}\left(\frac{16.1}{23.3}\right)^4\right] = 1110 \text{ lb. per sq. in.}$$

Allowable load $= 1110 \times 9.5 \times 9.5 = 100{,}000$ lb.

Assume that the column supports a 10 by 14 in. girder and a cast iron pintle which acts as the base for a 10 by 10 in. column 20 ft. long. Using a working stress in compression of 12,000 lb. per sq. in. for the pintle, find its dimensions.[1]

[1] "Design of Modern Steel Structures," by L. E. Grinter, The MacMillan Co., 1941.

Load on pintle:

$$L/d \text{ of column} = \frac{20 \times 12}{9.5} = 25.3.$$

Since L/d is greater than K the formula for long columns applies.

$$\frac{P}{A} = \frac{0.274E}{(L/d)^2} = \frac{0.274 \times 1{,}600{,}000}{25.3^2} = 685 \text{ lb. per sq. in.}$$

Allowable load $= 685 \times 9.5 \times 9.5 = 61{,}750$ lb.

$$\text{Required area of pintle} = \frac{61{,}750}{12{,}000} = 5.15 \text{ sq. in.}$$

Try a pintle with a stem having an outside diameter of 3 in. and an inside diameter of 1½ in.

$$\text{Area} = \frac{\pi}{4}(3^2 - 1.5^2) = 5.3 \text{ sq. in.}$$

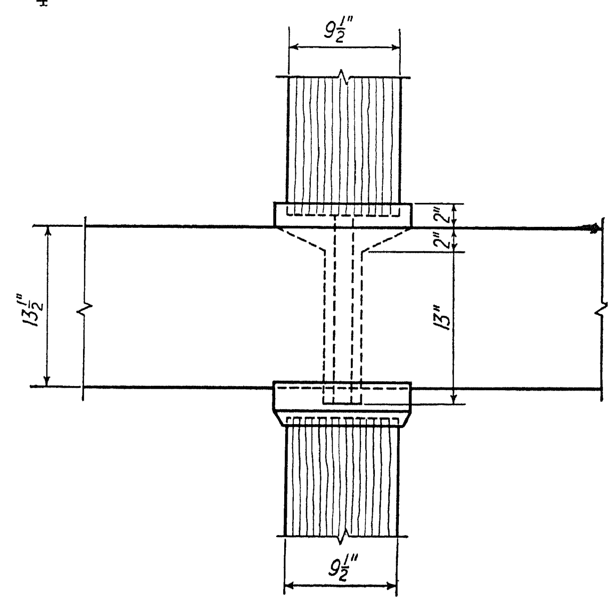


FIG. 46. Column supported on cast iron pintle.

64. Members with Combined Bending and Axial Loads. The design of members subjected to both axial tension and bending is limited by the formula $f = P/A + Mc/I$. However, special formulas are necessary for the design of compression members subjected to axial and bending loads because the allowable compressive stress differs from the allowable stress in tension.

The following formulas for columns with end loads, side loads, and eccentricity have been derived by J. A. Newlin, specialist in the me-

chanics of timber, Forest Products Laboratory, and have been published in *Building Standards Monthly*, December, 1940. They have also been published by the Timber Engineering Company in a document by J. E. Myer.

SYMBOLS USED

f = Allowable extreme fiber stress in bending in pounds per square inch.

L = Length of span or column in inches.

d_b = Dimension of a rectangular column in the direction of the side load.

e = Eccentricity in inches.

S_b = M/S = Unit bending stress due to side load.

k = The ratio of the unit bending stress due to side load to the unit compressive stress due to end load when side load is proportional to end load.

c_b = Allowable stress in compression parallel to the grain, in pounds per square inch, that would be permitted for the column if axial compression stress only existed, i.e., the safe working stress for the appropriate L/d_b ratio.

Q = Allowable stress in compression parallel to grain, in pounds per square inch, that is permitted for the column when side and/or eccentric load is combined with end load.

COLUMNS WITH $L/d_b = 10$ OR LESS

Combined end and side load:

$$Q = \frac{c_b(f - S_b)}{f}$$

Eccentrically loaded columns:

$$Q = \frac{c_b f}{f + c_b\left(\frac{6e}{d_b}\right)}$$

Side load proportional to end load:

$$Q = \frac{c_b f}{f + c_b k}$$

Combined end load, side load, and eccentricity:

$$Q = \frac{c_b(f - S_b)}{f + c_b\left(\frac{6e}{d_b} + k\right)}$$

COLUMNS WITH L/d_b = 20 OR MORE

Combined end and side load:

$$Q = \frac{f + c_b}{2} - \sqrt{\left(\frac{f + c_b}{2}\right)^2 - c_b(f - S_b)}$$

Eccentrically loaded:

$$Q = \frac{f + c_b\left(1 + \frac{15e}{2d_b}\right)}{2} - \sqrt{\left(\frac{f + c_b\left(1 + \frac{15e}{2d_b}\right)}{2}\right)^2 - c_b f}$$

Side load proportional to end load:

$$Q = \frac{f + c_b(1 + k)}{2} - \sqrt{\left(\frac{f + c_b(1 + k)}{2}\right)^2 - c_b f}$$

Combined end load, side load, and eccentricity:

$$Q = \frac{f + c_b\left(1 + \frac{15e}{2d_b} + k\right)}{2} - \sqrt{\left(\frac{f + c_b\left(1 + \frac{15e}{2d_b} + k\right)}{2}\right)^2 - c_b(f - S_b)}$$

COLUMNS WITH L/d_b BETWEEN 10 AND 20

When L/d_b is between 10 and 20 the stress should be assumed to vary as a straight line. The value of Q computed by the formula for $L/d_b = 20$ will be found to be only slightly less than the value found by interpolation between the two formulas, and for all practical purposes it will usually be unnecessary to calculate Q for $L/d_b = 10$ and then interpolate.

If the top chord of a roof truss supports roof rafters between the panel points, it is subjected to combined axial compression and side loading. If the top chord is continuous through the panel point, the bending moment may be computed from the formula $WL/10$. If a splice occurs within a span, the same bending moment, $WL/10$, may be used for this span also.

Example. As a specific example of the design of a top chord member of a truss assume the following:

Flat top truss, panel length 8 ft.
Truss spacing 16 ft. on centers.
Uniformly loaded top chord.
Total dead and live loads, 40 lb. per sq. ft.
Load in top chord, 20,000 lb. compression.

Working stresses:

Flexure, 1400 lb. per sq. in.
Compression, 1000 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Assume a member 5½ by 11½ in.

$$\frac{L}{d_b} = \frac{96}{11.5} = 8.35$$

This ratio is less than 10, so the following formula applies:

$$Q = \frac{c_b f}{f + c_b k}$$

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{40 \times 16 \times 8 \times 96}{10 \times 124}}{\frac{20{,}000}{5.5 \times 11.5}} = \frac{396}{316} = 1.25$$

$$Q = \frac{1000 \times 1400}{1400 + 1000 \times 1.25} = 528 \text{ lb. per sq. in.}$$

Since the actual load of 316 lb. per sq. in. is less than the computed permissible load of 528 lb. per sq. in., the column is adequate.

Example. As an example illustrating the method used in determining the adequacy of a column whose L/d_b ratio is between 10 and 20, assume the same conditions as those given in the previous example, but consider the panel length as 12 ft.

$$\frac{L}{d_b} = \frac{144}{11.5} = 12.5$$

One of the following formulas applies:

$$\frac{L}{d_b} = 20 \text{ or more}$$

$$Q = \frac{f + c_b(1 + k)}{2} - \sqrt{\left(\frac{f + c_b(1 + k)}{2}\right)^2 - c_b f}$$

$$\frac{L}{d_b} = 10 \text{ or less}$$

$$Q = \frac{c_b f}{f + c_b k}$$

If the formula for an L/d_b ratio equal to 20 or more is used and the load-carrying capacity of the member is found sufficient, no further computations are necessary.

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{40 \times 16 \times 12 \times 144}{10 \times 124}}{\frac{20{,}000}{5.5 \times 11.5}} = \frac{893}{316} = 2.83$$

$c_b = 981$ lb. per sq. in., from column formula.

$$Q = \frac{1400 + 981(1 + 2.83)}{2} - \sqrt{\left(\frac{1400 + 981(1 + 2.83)}{2}\right)^2 - 981 \times 1400}$$

$$= 2577 - 2270 = 307 \text{ lb. per sq. in.}$$

The permissible stress of 307 lb. per sq. in. is very little less than the stress developed, so the member may be considered adequate.

PROBLEMS

1. Design a timber beam simply supported at its ends to carry a uniform load of 400 lb. per ft. and concentrated loads of 3000 lb. and 2000 lb. at the quarter points when the span is 16 ft. Use 1600#f close-grained Douglas fir.

2. If 3 by 10 in. joists span 17 ft. and are placed 24 in. on centers, what is the maximum safe load in pounds per square foot that can be placed on the floor? Use No. 1 dense structural southern pine for the joists and 1-in. hemlock sub-flooring with a finish floor of 1-in. oak.

3. Design a beam to span 10 ft. and carry 25,000 lb. concentrated at the center. Check shear and deflection, and design the bearing plates at the reaction. Use 1400#f close-grained redwood.

4. An 8 by 10 in. timber beam with a span of 12 ft. supports a concentrated load P at the mid-span. The beam is No. 1 structural longleaf pine. Allowing for the weight of the beam, what is the maximum safe value of P?

5. Using bolts and connectors, and 10 by 10 in. timbers, design a built-up beam to carry 1000 lb. per ft. over a span of 20 ft. Include the weight of the beam and use close-grained Douglas fir.

6. A 6 by 6 in. column is 10 ft. long. What safe load will it carry, if the column is tidewater red cypress with a working value in compression of 1200 lb. per sq. in.?

7. Design a column 16 ft. long to carry a load of 100,000 lb. Use select structural Douglas fir.

8. A round column 12 ft. high supports an axial load of 12,000 lb. Using close-grained redwood, what size column is required?

9. Determine the load-carrying capacities of the following columns: (*a*) 12 by 12 in. by 11 ft. 0 in. 1100#c Douglas fir; (*b*) 8 by 10 in. by 16 ft. 0 in. 1200#c southern cypress; (*c*) 6 by 10 in. by 14 ft. 0 in. 1100#c oak; (*d*) 10 by 12 in. by 18 ft. 0 in. 1000#c dense shortleaf southern pine.

10. Design a 16 ft. 0 in. column of 1000#c dense shortleaf southern pine to carry an axial load of 12,120 lb. and a side load of 20 lb. per sq. ft. The columns are spaced 16 ft. 0 in. on centers.

CHAPTER V

WOOD TRUSSES

65. Types of Trusses. Figure 47 illustrates the types of wood trusses most commonly used in building construction. These are usually built of wood throughout with the exception of the metal used as bolts and connectors in framing the joints and splices. The top and bottom chords and the diagonal tension members are made up of two or more spaced members with the web compression member as a solid piece. The bowstring trusses often have a two-member top chord consisting of relatively thin pieces of lumber glued together to form the required thickness.

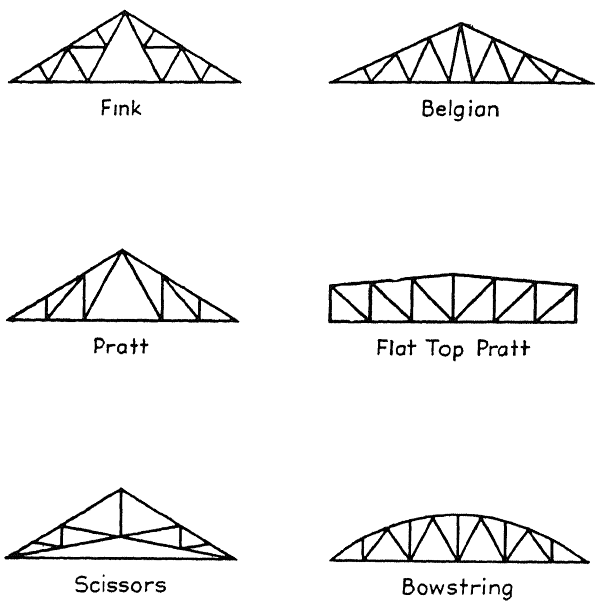


FIG. 47. Types of wood trusses.

66. Design Procedure. In the design of a wood truss there are certain chronological steps that when followed will reduce the amount of work involved. The general procedure is outlined below:

1. Determine the live loads acting on the truss.

2. Compute the dead loads, including roofing, sheathing, rafters, purlins, and weight of truss and any other stationary loads.

3. Determine the stresses in the members.

4. Select the species and grade of lumber to be used. This selection depends upon the locality in which the structure is built and the relative cost of the various species that are usable for structural purposes.

5. Compute the size of each member required to carry the stress due to the combination of loads causing the greatest stress.

6. Select the size and type of connector. When possible, it is best to keep the same size and type throughout the truss.

7. Design the joint that carries the greatest load. The placement of the rings in this joint will usually determine the size of the members and will facilitate the design of the remaining joints. Determine the member in the joint that has the loads acting on it at an angle to the grain and design for the largest load first.

8. Calculate the permissible reductions in the spacing of connectors and the end distances of the members.

9. Check the edge distance of the connectors and the size of the members.

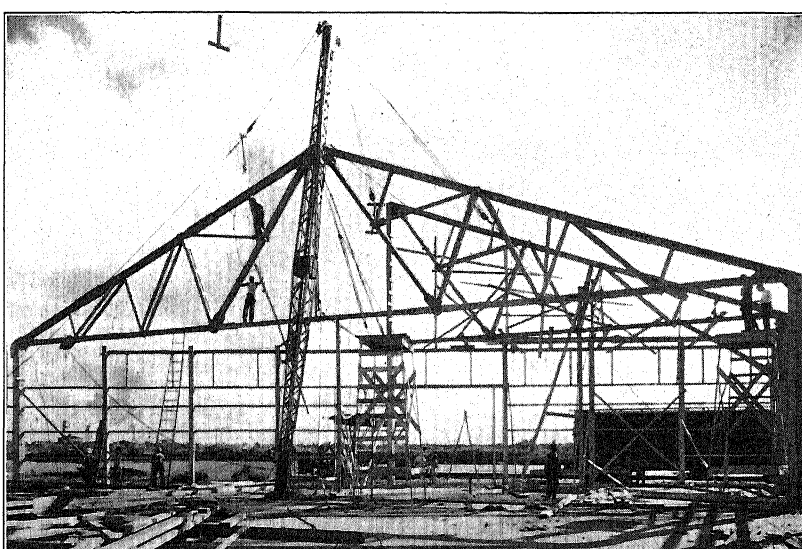


FIG. 48. Erection of 100-ft. Fink trusses.

67. Dead and Live Loads. The total load for which a truss must be designed consists of the weight of the roofing material, rafters, purlins, the weight of the truss, the snow load, and the pressure of the wind. The

following table may be used in estimating the weights in pounds per square foot of roof surface of various roofing material.

Table 23. Weights of Roofing Material

Roofing Material	*Approximate Weight (lb. per sq. ft.)*
Corrugated asbestos, $\frac{1}{4}$ in. thick	3
Corrugated asbestos, $\frac{3}{8}$ in. thick	4.5
Corrugated steel, No. 22 gage (U. S. Standard)	1.5
Corrugated steel, No. 20 gage (U. S. Standard)	1.8
Corrugated steel, No. 18 gage (U. S. Standard)	2.3
Felt and asphalt	2
Slag roof, 4-ply with cement and sand	4
Slate, $\frac{1}{8}$ in. thick, 3-in. double lap	4.5
Slate, $\frac{3}{16}$ in. thick, 3 in. double lap	6.75
Tiles	10–20
Wood, shingles	2

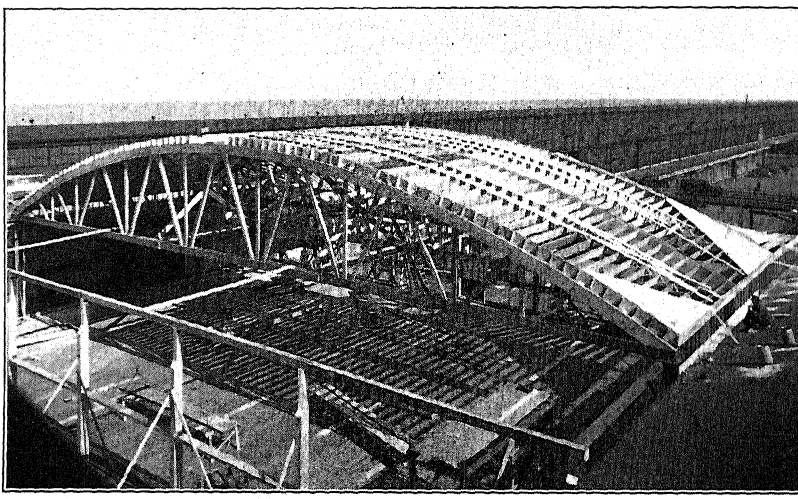


FIG. 49. 111-ft. bowstring trusses designed and built by McKeown Bros., Chicago, Ill.; 2½-in. split rings used between web members and chords, and 4-in. flange shear plates at heel joints.

The weights of wood in pounds per 1000 board feet given in Table 24 are based on the nominal size; and in estimating the weight of any given quantity of lumber, it is necessary to multiply the weights in the table by the ratio of the actual size of each piece to the nominal size. For instance, to find the actual weight of 1000 board feet of larch 2 by 4 in. boards, multiply 3000 by the ratio $\frac{1.625 \times 3.625}{2 \times 4}$. The result is 2210 lb.

Table 24. Average Weights of Various Species of Wood

Species	Weight, lb. per cu. ft. Green	Weight, lb. per cu. ft. *Air-Dry* (12% moisture content)	Weight, lb. per 1000 bd. ft. (Nominal Size) (12% moisture content)
Ash, white	48	41	3420
Beech	54	45	3750
Birch	57	44	3670
Cedar, western red	27	23	1920
Chestnut	55	30	2500
Cypress, southern	51	32	2670
Douglas fir (coast region)	38	34	2830
Douglas fir (inland region)	36	31	2580
Elm, rock	53	44	3670
Gum, black	45	35	2920
Gum, red	50	34	2830
Gum, tupelo	56	35	2920
Hemlock, eastern	50	28	2330
Hemlock, western	41	29	2420
Hickory, pecan	62	45	3750
Hickory, true	63	51	4250
Larch, western	48	36	3000
Maple, hard	56	44	3670
Oak, red	64	44	3670
Oak, white	63	47	3920
Pine, Norway	42	34	2830
Pine, longleaf	55	41	3420
Pine, shortleaf	52	36	3000
Poplar, yellow	38	28	2330
Redwood	50	28	2330
Spruce, eastern	34	28	2330
Spruce, Sitka	33	28	2330
Walnut, black	58	38	3170

The weights of wood trusses in pounds per square foot of horizontal surface may be found by using the following formulas developed by the writer:

Pitched roof trusses	$0.064S$
Flat top Pratt trusses	$0.043S + 1.75$
Bowstring trusses	$0.038S + 0.6$

S is the span of the truss, in feet.

The snow load, which depends upon the location and the roof pitch of the structure, ranges from practically 0 to 50 lb. per sq. ft. of roof surface. For roofs having a ⅓ to ¼ pitch, a snow load of 15 to 20 lb. per sq. ft. is commonly used in the Central States and 5 to 10 lb. per sq. ft.

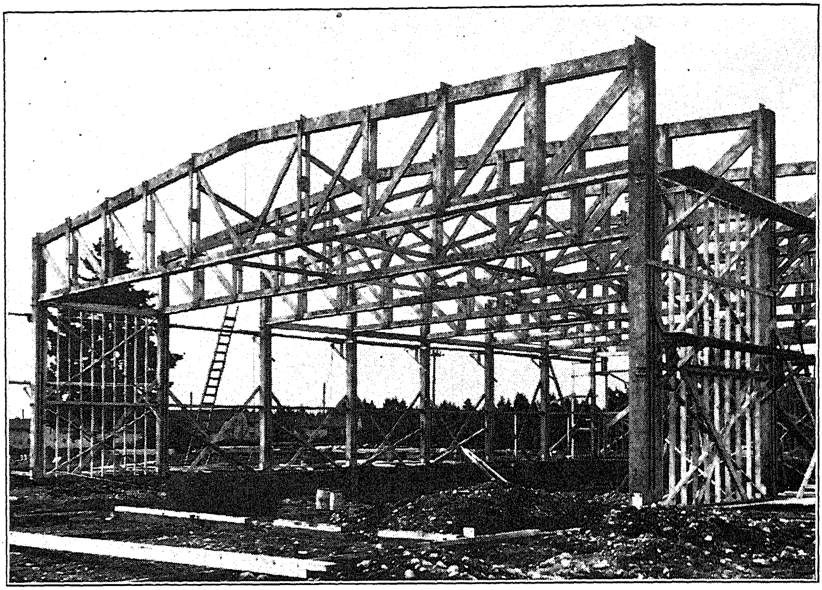


Fig. 50. 75-ft. Pratt trusses with spaced columns made an integral part of the truss.

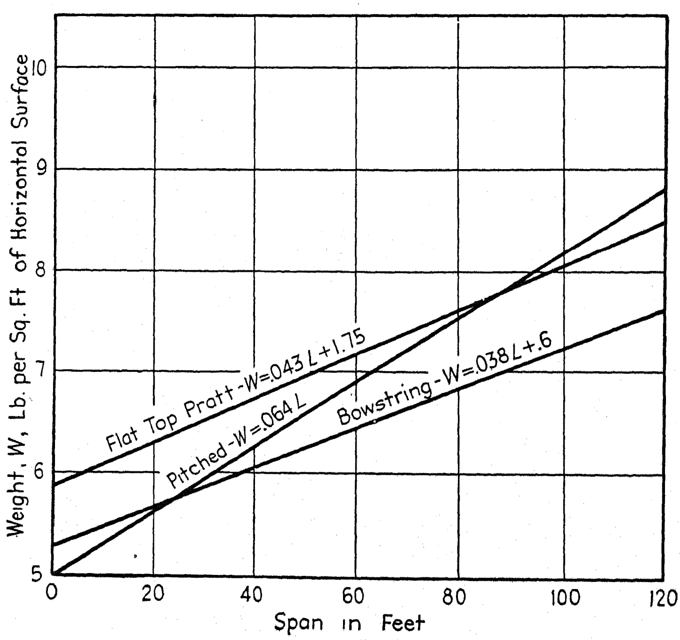


Chart 14. Curves for estimating weights of wooden roof trusses.

in the Southern States. For flat roofs these values should be doubled, and for steep roofs they may be decreased by 50 per cent.

The wind loads vary throughout the country. Most building codes specify the amount to use in designing a structure. The most common value is 20 lb. per sq. ft. of vertical surface. The component of the wind load normal to the roof may be found by Duchemin's formula.[1]

$$P_n = P_h\left(\frac{2\sin\theta}{1+\sin^2\theta}\right)$$

P_h = horizontal pressure per square foot on a vertical surface.
P_n = normal pressure per square foot on a sloping surface.
θ = angle of inclination of the roof surface.

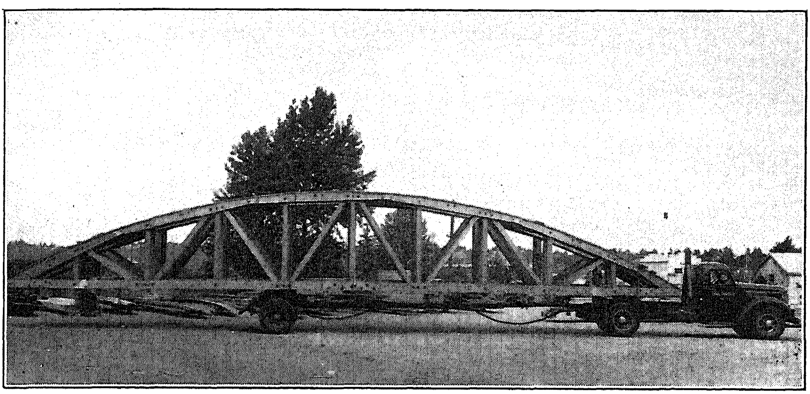


FIG. 51. Modified arch type bowstring truss being transported by specially built truck.

68. Design of a 50-Ft. Fink Truss. A 50-ft. wooden Fink truss is to be designed to carry 6 by 10 in. Douglas fir purlins at the panel points which

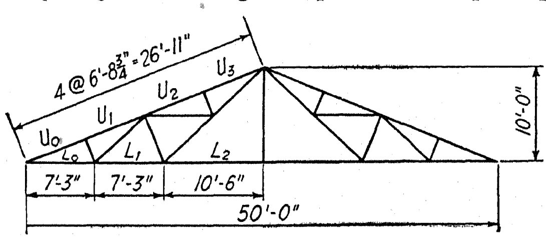


FIG. 52. 50-ft. Fink truss.

support 2-in. T. & G. southern pine sheathing and 3/16-in. slate roofing. The trusses are spaced 16 ft. on centers and have the dimensions as shown in Figure 52.

[1] See Appendix A.

Dead Loads

Weight of truss per square foot of horizontal surface $= 0.064 \times 50 = 3.2$ lb.
Total weight of truss $= 3.2 \times 50 \times 16 = 2560$ lb.

Load per panel point $= \dfrac{2560}{8} = 320$ lb.

Weight of purlins $= \dfrac{(5.5 \times 9.5)^2 \times 16 \times 2830}{6 \times 10 \times 12 \times 1000} = 172$ lb., use 200 lb.
Load per panel point $= 200$ lb.
Weight of sheathing $= 1.625 \times 3 = 4.87$ lb. per sq. ft.
Load per panel point $= 4.87 \times 16 \times 6.73 = 524$ lb., use 530 lb.
Weight of roofing $= 6.75$ lb. per sq. ft.
Load per panel point $= 6.75 \times 16 \times 6.73 = 727$ lb.

Live Loads

Snow load $= 20$ lb. per sq. ft.
Load per panel point $= 20 \times 16 \times 6.73 = 2150$ lb.
Wind load $= 20$ lb. per sq. ft., horizontal:

$$P_n = 20\left(\frac{2 \sin 22°}{1 + \sin^2 22°}\right) = 13.10 \text{ lb. per sq. ft.}$$

Load per panel point $= 13.1 \times 16 \times 6.73 = 1400$ lb.

	Total Panel Loads
Slate	730
Sheathing	530
Purlins	200
Truss	320
Total dead panel load	1780
Snow	2150
Wind	1400

Figure 53 is the dead load stress diagram. The stresses due to snow load may be calculated by multiplying the dead load stresses by the ratio $\dfrac{2150}{1780}$.

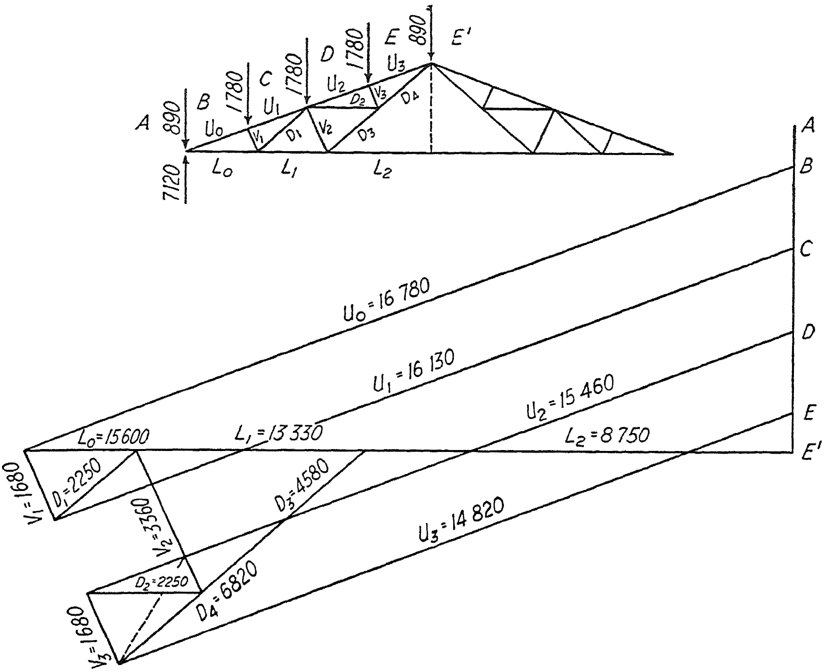


FIG. 53. Dead load stress diagram.

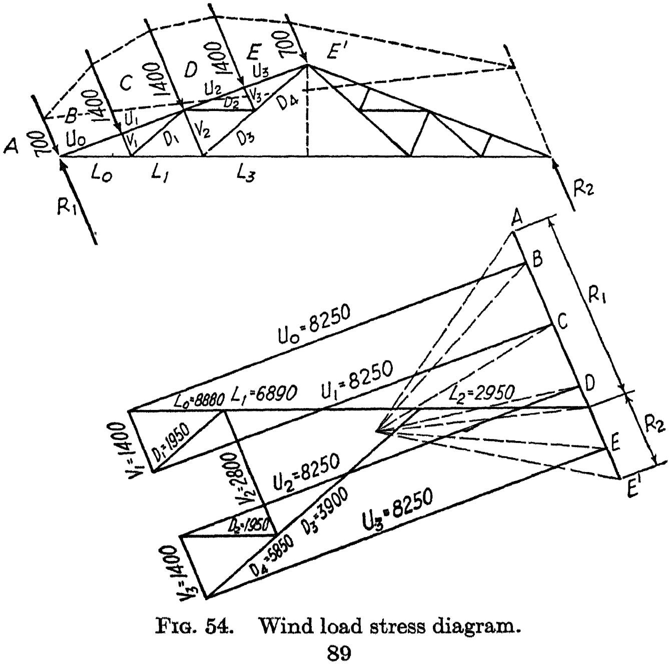


FIG. 54. Wind load stress diagram.

Maximum Stresses

In designing members subjected to dead, live, and wind loads there are two combinations to be considered: (1) dead load plus live load, (2) dead load plus live load plus wind load. When designing for dead load plus live load, the full allowable working stresses for the respective structural grades of lumber are used. When designing for a combination of wind load plus dead and live loads, the allowable working stresses may be increased by 50 per cent (except modulus of elasticity) providing the size of the member required is not less than that obtained when considering dead and live loads only.

Table 25. Maximum Stresses in Members

Member	*Dead Load*	*Snow Load*	*Wind Load*	*Maximum Dead Load + Snow Load*	*Maximum Dead + Snow + Wind Load*
U_0	−16,780	−20,250	−8,250	−37,030	−45,280
U_1	−16,130	−19,500	−8,250	−35,630	−43,880
U_2	−15,460	−18,690	−8,250	−34,150	−42,400
U_3	−14,820	−18,020	−8,250	−32,840	−41,090
L_0	+15,600	+18,850	+8,880	+34,450	+43,330
L_1	+13,330	+16,110	+6,890	+29,440	+36,330
L_2	+ 8,750	+10,570	+2,950	+19,320	+22,270
V_1	− 1,680	− 2,030	−1,400	− 3,710	− 5,110
V_2	− 3,360	− 4,060	−2,800	− 7,420	−10,220
V_3	− 1,680	− 2,030	−1,400	− 3,710	− 5,110
D_1	+ 2,250	+ 2,720	+1,950	+ 4,970	+ 6,920
D_2	+ 2,250	+ 2,720	+1,950	+ 4,970	+ 6,920
D_3	+ 4,580	+ 5,550	+3,900	+10,130	+14,030
D_4	+ 6,820	+ 8,240	+5,850	+15,060	+20,910

Design of Members

Lumber grade: No. 1 dense structural southern pine.

Working stresses:

Tension, 1400 lb. per sq, in.
Compression parallel to grain, 1000 lb. per sq. in.
Compression perpendicular to grain, 380 lb. per sq. in.
Horizontal shear, 100 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Working stresses increased for wind load:

Tension, 2100 lb. per sq. in.
Compression parallel to grain, 1500 lb. per sq. in.
Compression perpendicular to grain, 570 lb. per sq. in.
Horizontal shear, 150 lb. per sq. in.
Modulus of elasticity, 1,600,000 lb. per sq. in.

Top chord: Use two spaced members. Length of column = 6 ft. $8\frac{3}{4}$ in.

Case I. Maximum load due to dead and live loads = 37,030 lb.
Case II. Maximum load due to dead, live, and wind loads = 45,280 lb.

CASE I. Assuming a thickness of $2\frac{5}{8}$ in., the L/d ratio becomes 31.

$$K = 0.64\sqrt{\frac{2.5 \times 1{,}600{,}000}{1000}} = 40.5$$

The member is in the intermediate class and the allowable working stress $= 1000\left[1 - \frac{1}{3}\left(\frac{80.75}{40.5 \times 2.625}\right)^4\right] = 890$ lb. per sq. in.

$$\text{Required area} = \frac{37{,}030}{890} = 41.7 \text{ sq. in.}$$

Two 3 by 10 in. pieces with a total area of 49.88 sq. in. will satisfy.

CASE II. $K = 0.64\sqrt{\dfrac{2.5 \times 1{,}600{,}000}{1500}} = 33.6$

$$\text{Allowable working stress} = 1500\left[1 - \frac{1}{3}\left(\frac{80.75}{33.6 \times 2.625}\right)^4\right] = 1140 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{45{,}280}{1140} = 39.8 \text{ sq. in.}$$

This area is less than that required to carry the dead and live loads, and two 3 by 10 in. pieces will be used for the top chord unless it is found necessary in the design of the joints to increase the size.

Bottom chord:

Case I. Maximum load = 34,450 lb.
Case II. Maximum load = 43,330 lb.

The thickness of the bottom chord is made the same as for the top chord so that the splice plates at the heel joint will lie flat on the faces of both chords. By using the same thickness, the compression web members can be single pieces placed between the chord members, and the diagonal tension members can be made up of two pieces placed outside of the chord members without using fillers at the joints.

$$\text{Case I.} \quad \text{Required area} = \frac{34{,}450}{1400} = 24.6 \text{ sq. in.}$$

$$\text{Case II.} \quad \text{Required area} = \frac{43{,}330}{2100} = 20.6 \text{ sq. in.}$$

Two 3 by 6 in. pieces with a total area of 29.54 sq. in. will carry the load, but it will be found that this size is inadequate to carry the joint stresses.

Web members:

The size of the web members depends upon the design of the joints rather than on the load to be carried in each member. If 4-in. split rings are selected, the minimum width of the web members is 6 in. From the table of maximum stresses it is apparent that this width is ample providing the thickness is great enough.

The web members in compression will be designed as solid columns and will all have the same thickness. Member V_2 has the greatest load and a length of 5 ft. 4⅝ in. Using a 3-in. width, the L/d ratio is 25, and the working stress becomes 700 lb. per sq. in.

Required area = 10.6 sq. in. A 3 by 6 in. member has an area of 14.77 sq. in.

D_4 has the greatest load for the diagonal tension members.

$$\text{Required area} = \frac{15{,}060}{1400} = 10.75 \text{ sq. in.}$$

Two 2 by 6 in. pieces have an area of 18.28 sq. in.

Design of Joints

Lower chord joint:

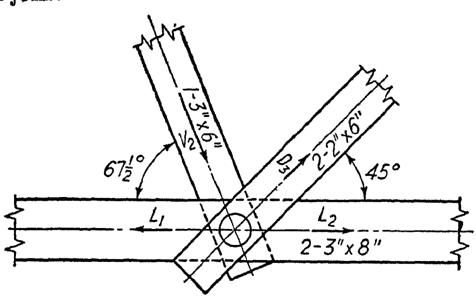


Fig. 55. Lower chord joint for 50-ft. Fink truss.

Case I. The two tension members, D_3, have their load of 10,130 lb. acting on the chord L_1L_2 at an angle of 45°. At this angle a 4-in. split ring will carry 5355 lb.

$$\text{No. of rings required} = \frac{10{,}130}{5355} = 2, \text{ one between each chord member}$$

and diagonal.

The load is acting upward and the compression side of the ring in L_1L_2 is measured vertically from the center of the ring to the upper side of the piece.

The standard edge distance is $3\frac{3}{4}$ in., but the full capacity of the rings is not developed, and this edge distance can be reduced.

$$\text{Capacity of rings developed} = \frac{10{,}130}{5355 \times 2} = 94.5\%$$

Reduced edge distance $= 3\frac{3}{8}$ in.

This requires a chord 8 in. (nominal) wide.

The compression member V_2 transmits its load of 7420 lb. to the chord at an angle of $67\frac{1}{2}°$. At this angle a 4-in. split ring has a load-carrying capacity of 4880 lb.

No. of rings required $= 2$, one between each chord member and V_2.

The compression side of the ring in L_1L_2 is measured to the bottom of the piece.

Capacity of rings developed $= \dfrac{7440}{9760} = 76.2\%$. Minimum edge distance $= 2\frac{3}{4}$ in.

Because the edge distance on the other side of the ring determined the width of the member it is not possible to take advantage of the above reduction.

The standard end distance in D_3 is 7 in.

Capacity of 4-in. ring when load is parallel to grain $= 6400$ lb.

Reduced end distance $= 5\frac{1}{8}$ in.

The standard end distance in V_2 is $5\frac{1}{2}$ in.

Capacity of 4-in. ring when load is parallel to grain $= 6300$ lb.

Reduced end distance $= 3\frac{1}{4}$ in.

Case II. When designing for a combination of dead, live, and wind loads, the load-carrying capacities of split rings may be increased by 30 per cent.

Member D_3, No. of rings $= \dfrac{14{,}030}{5355 \times 1.30} = 2$. The result is slightly greater than 2 but not enough to make it necessary to use 4 rings. However, no reduction in edge distance is permissible, but none is necessary because the width of the chord has been established as 8 in.

$$\text{End distance, capacity of rings developed} = \frac{14{,}030}{6400 \times 1.30 \times 2} = 85\%$$

End distance $= 5\frac{5}{8}$ in.

$$\text{Member } V_2\text{, No. of rings} = \frac{10{,}220}{4880 \times 1.3} = 2$$

$$\text{End distance, capacity of rings developed} = \frac{10{,}220}{6300 \times 1.3 \times 2} = 63\%$$

End distance $= 3\frac{3}{8}$ in.

The only change required in the joint designed under Case I is the minimum end distance in members D_3 and V_2.

Heel joint:

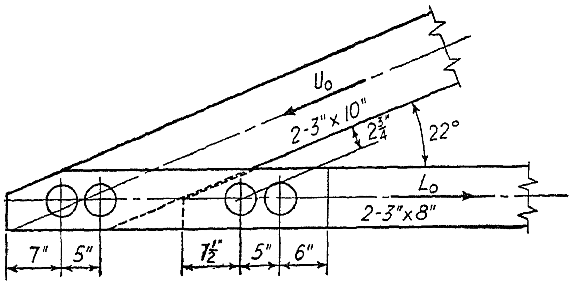


Fig. 56. Heel joint for 50-ft. Fink truss.

Member U_0 transmits the horizontal component of its load through the wood splice plates to member L_0. This component acts at an angle of 22° in member U_0, and the number of rings required in this part of the joint is determined by their capacity at this angle. The angle is less than 30° and the rings may be spaced with reference to either member; i.e., U_0 or the splice plates.

Capacity of 4-in. split ring when load is applied at 22° and rings are opposite in both faces of $2\frac{5}{8}$-in. lumber = 5835 lb.

$$\text{No. of rings required, Case I} = \frac{34{,}450}{5835} = 5.9$$

There are 4 rings required for each bolt, and in order to keep the joint symmetrically loaded 8 rings will be used.

$$\text{No. of rings required, Case II} = \frac{43{,}330}{5835 \times 1.3} = 5.7$$

The rings will be spaced parallel to the grain in the splice plates and the standard spacing is 9 in.

$$\text{Ring capacity developed} = \frac{34{,}450}{5835 \times 8} = 74\%$$

Capacity developed for the two groups of rings = 74 × 2 = 148%

Letting one group carry 100% of its capacity, the other group is only carrying 148 − 100 = 48% of its capacity. At this percentage the spacing between the two groups can be reduced to $4\frac{7}{8}$ in. or 5 in. The

end distance for the splice plates is taken as 7 in. and for member U_0 as 5½ in. The minimum edge distance for both members is 2¾ in.

The load between the bottom chord, L_0, and the splice plates is 34,450 lb. and acts parallel to the grain.

$$\text{No. rings required, Case I} = \frac{34{,}450}{6300} = 5.5$$

$$\text{No. rings required, Case II} = \frac{43{,}330}{6300 \times 1.3} = 5.3$$

To keep the joint symmetrically loaded 8 rings will be used.

$$\text{Ring capacity developed} = \frac{34{,}450}{6300 \times 8} = 68.5\%$$

Letting one group carry 100% of its capacity, the other group is developing only $68.5 \times 2 - 100 = 37\%$ of its capacity.

Reduced spacing = 5 in. This represents a reduction in load-carrying capacity of 50%.

Percentage left for reducing end distance $= 200 - 68.5 \times 2 - 50 = 13\%$.

Make end distance = 6 in.

The edge distance measured from the diagonal cut in L_0 to the center of the ring must be 2¾ in.

Peak joint:

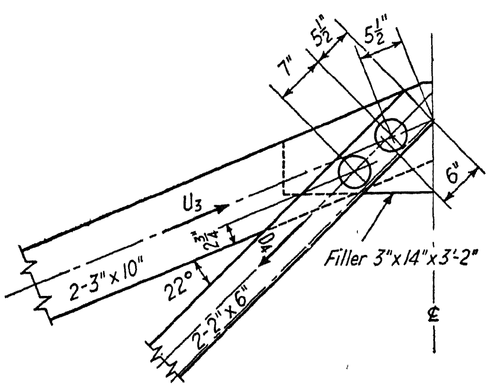


FIG. 57. Peak joint for 50-ft. Fink truss.

At the peak joint it is necessary to use a filler having the same thickness as the compression web members to transmit the horizontal component to the other half of the truss. This horizontal component acts on member U_3 at an angle of 22°, and the load in member D_4 also acts on U_3 at 22°.

Capacity of a 4-in. split ring when load acts at an angle of 22° = 5835 lb.

No. of rings required, Case I, horizontal component $= \dfrac{19{,}320}{5835} = 3.3$. Use 4.

No. of rings required, Case II, horizontal component $= \dfrac{22{,}270}{5835 \times 1.3} = 3$. Use 4.

No. of rings required, Case I, D_4 to $U_3 = \dfrac{15{,}060}{5835} = 2.6$. Use 4.

No. of rings required, Case II, D_4 to $U_3 = \dfrac{20{,}910}{5835 \times 1.3} = 2.8$. Use 4.

The end distance in member U_3 is taken as standard at 5½ in.

The end distance in member D_4 is reduced to 5½ in.

Capacity of rings developed $= \dfrac{19{,}320}{5835 \times 4} = 83\%$

Reduced spacing = 7 in.

The end distance in member U_3 locates the gage line for the end connectors. This end distance is measured slightly back from the extreme end of the member.

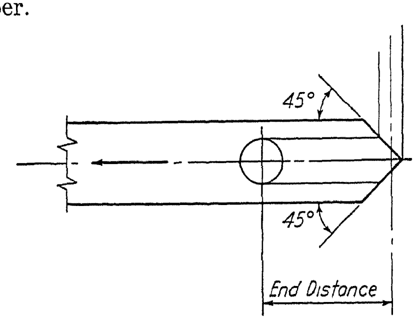


FIG. 58. Method of measuring end distance for members with diagonal end cuts at 45°.

In order to maintain the required edge distances for all members and still locate the connectors within the limits of D_4, it is necessary to offset D_4 from the panel point. This slight eccentricity will not change the design and may be disregarded.

69. Design of an 80-Ft. Fink Truss. Figure 59 illustrates one-half of an 80-ft. Fink truss. The loads in the members are based on a combined

dead and live load of 40 lb. per sq. ft. of horizontal surface with the trusses 16 ft. on centers. Use the same grade of lumber and working stresses as used in the previous example.

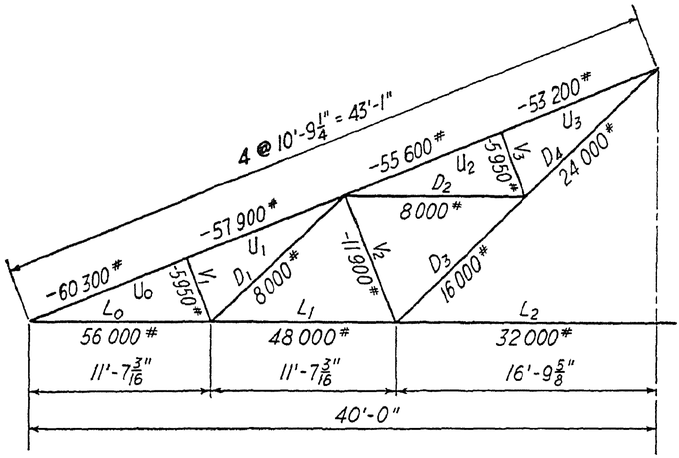


FIG. 59. One-half of 80-ft. Fink truss.

DESIGN OF MEMBERS

Top chord:

Length = 10 ft. 9¼ in. Select a timber 4 in. in nominal thickness.

$$L/d = \frac{129.25}{3.625} = 35.7$$

From the spaced column formula, $\frac{P}{A} = c\left[1 - \frac{1}{3}\left(\frac{L}{K_2 d}\right)^4\right]$, the working stress becomes 799 lb. per sq. in.

$$\text{Area required} = \frac{60{,}300}{799} = 75.5 \text{ sq. in.}$$

Two 4 by 12 in. timbers with a cross-sectional area of 83.38 sq. in. will satisfy the requirements.

Bottom chord:

$$\text{Required area} = \frac{56{,}000}{1400} = 40 \text{ sq. in.}$$

Two 4 by 6 in. members with a cross-sectional area of 40.78 sq. in. will satisfy the requirements, but the joint between L_1 and L_2 will probably require a larger width.

Web members:

Select a member 4 in. in nominal thickness for the web members in compression. These will be designed as solid columns.

V_2 has a length of 103.375 in.

$$L/d = \frac{103.375}{3.625} = 28.5$$

$$K = 0.64\sqrt{\frac{1,600,000}{1000}} = 25.6$$

$$\frac{P}{A} = \frac{0.274E}{(L/d)^2} = \frac{0.274 \times 1,600,000}{(28.5)^2} = 540 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{11,900}{540} = 22 \text{ sq. in.}$$

A 4 by 8 in. member with a cross-sectional area of 27.19 sq. in. will be satisfactory. However, the lower chord joint may require a larger width.

Using a 4-in. nominal or 3⅝-in. actual thickness for compression members V_1 and V_3, which corresponds in thickness to V_2, the L/d ratio becomes $\frac{51.688}{3.625} = 14.24$. Since this L/d ratio is greater than eleven but less than K, the formula for intermediate columns is used in calculating the allowable working stress.

$$\frac{P}{A} = 1000\left[1 - \frac{1}{3}\left(\frac{14.24}{25.6}\right)^4\right] = 968 \text{ lb. per sq. in.}$$

$$\text{Required area} = \frac{5950}{968} = 6.15 \text{ sq. in.}$$

If 4-in. split rings are used the minimum width permitted is 5½ in. This requires a 4 by 6 in. member with a cross-sectional area of 20.39 sq. in.

The maximum load for the diagonal tension members is in member D_4.

$$\text{Required area} = \frac{24,000}{1400} = 17.13 \text{ sq. in.}$$

Two 2 by 8 in. members with a cross-sectional area of 20.39 sq. in. are required.

For members D_1 and D_2 two 2 by 6 in. pieces will satisfy the requirements.

Design of Joints

Heel joint:

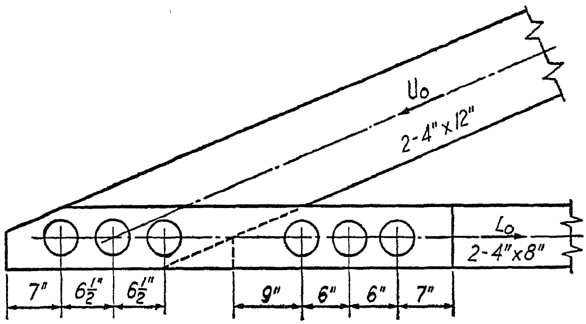


FIG. 60. Heel joint for 80-ft. Fink truss.

	Member U_0 and Splice Plates	Member L_0 and Splice Plates
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of piece	$3\frac{5}{8}$″	$3\frac{5}{8}$″
Angle of load to grain	22°	0°
Standard design load for one connector	5,930 lb.	6,400 lb.
Load on joint	56,000 lb.	56,000 lb.
Connectors required	9.45	8.75
Connectors used	12	12
Percentage of capacity developed	79%	73%
Standard spacing	9″	9″
Capacity developed of two groups with one group carrying 100% of its capacity = $\dfrac{79 \times 3 - 100}{2}$	68.5%	59.5%
Reduced spacing	$6\frac{1}{2}$″	6″
Edge distance required	$2\frac{3}{4}$″	$2\frac{3}{4}$″
End distance, splice plates	7″	7″

End distance, U_0, $5\frac{1}{2}$″

End distance, L_0, 9″

Lower chord joint:

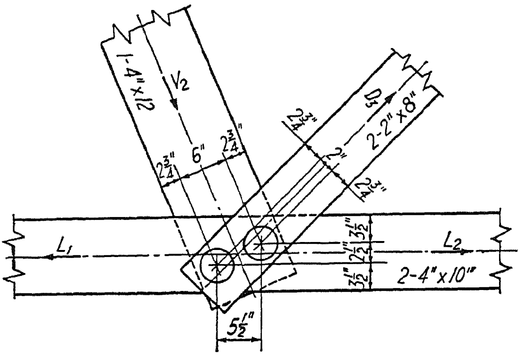


Fig. 61. Lower chord joint for 80-ft. Fink truss.

	V_2 to Chord	D_3 to Chord
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of piece	$3\frac{5}{8}''$	$3\frac{5}{8}''$
Angle of load to grain	$67\frac{1}{2}°$	45°
Standard design load for one ring	4,960 lb.	5,440 lb.
Load on joint	11,900 lb.	16,000 lb.
Connectors required	2.4	2.94
Connectors used	4	4
Percentage of capacity developed	60%	73.5%
Standard edge distance, compression side of chord	$3\frac{3}{4}''$	$3\frac{3}{4}''$
Edge distance reduced	$2\frac{3}{4}''$	$2\frac{3}{4}''$
Spacing required parallel to grain	$5\frac{1}{2}''$	$5\frac{1}{2}''$

Width of chord, $2\frac{3}{4} + 2\frac{1}{2} + 2\frac{3}{4} = 8''$. Use $9\frac{1}{2}''$.

Width of V_2, $2\frac{3}{4} + 6 + 2\frac{3}{4} = 11\frac{1}{2}''$. Use $11\frac{1}{2}''$.

Width of D_3, $2\frac{3}{4} + 2 + 2\frac{3}{4} = 7\frac{1}{2}''$. Use $7\frac{1}{2}''$.

Standard design load for one connector at 0°	6,400 lb.	6,400 lb.
Percentage of capacity developed at 0°	46.5%	62.5%

End distance, V_2 in compression, $3\frac{1}{4}''$

End distance, D_3 in tension, $3\frac{1}{2}''$

Peak joint:

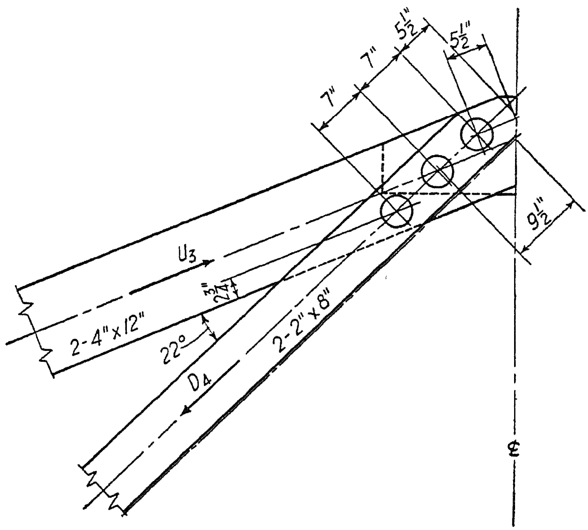


FIG. 62. Peak joint for 80-ft. Fink truss.

	D_4 to U_3	U_3 to Splice Plates
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of piece	$3\frac{5}{8}''$	$3\frac{5}{8}''$
Angle of load to grain	22°	22°
Standard design load for one connector	5,930 lb.	5,930 lb.
Load on joint	24,000 lb.	32,000 lb.
Connectors required	4.05	5.4
Connectors used	6	8
Percentage of capacity developed	67.5%	67.5%
Standard spacing required parallel to grain in D_4	9″	9″
Reduced spacing, $\dfrac{67.5 \times 3 - 100}{2} = 51\%$ of capacity for two groups	5″	
Reduced spacing, $\dfrac{67.5 \times 2 - 100}{2} = 35\%$ of capacity for one group		5″
Spacing used	7″	7″
End distance, D_4, $5\frac{1}{2}''$		
End distance, U_3, $5\frac{1}{2}''$		
Edge distance required	$2\frac{3}{4}''$	$2\frac{3}{4}''$

70. Design of a 60-Ft. Pratt Truss. In designing a flat-top Pratt truss it is not necessary to make the top and bottom chords the same thickness. Moreover, by using two spaced members for the chords, it is

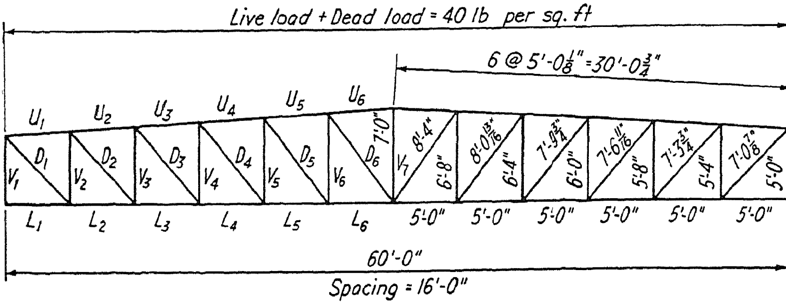


FIG. 63. A 60-ft. flat-top Pratt truss.

possible to place them on the outside of the joints. The vertical compression members can be treated as solid columns and the diagonal tension members as double members placed between the chord and vertical member. By arranging the joints in this manner the tension members become the second and fourth members in the joint and have

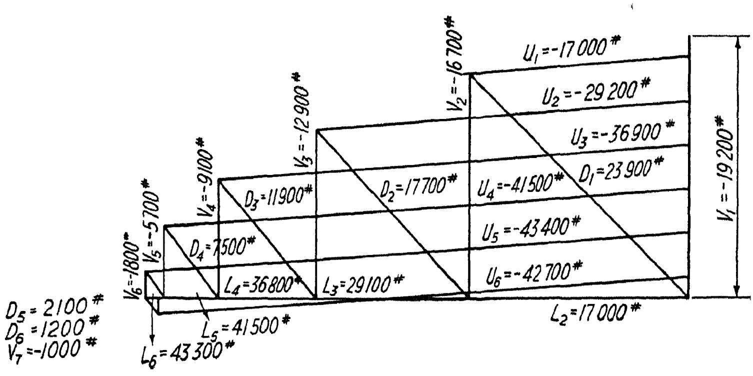


FIG. 64. Dead and live load stress diagram for 40 lb. per sq. ft. of horizontal surface.

the loads acting on them at an angle to their grain. The angles in this case will be smaller and the capacities of the connectors greater than a joint assembled with the chord members between the diagonal tension and vertical chord members.

As in the previous examples, dense No. 1 structural southern pine will be used for all members.

Top chord:

Length = 5 ft. $0\frac{1}{8}$ in. Select a timber 3 in. (nominal) in thickness.

$$L/d = \frac{60.01}{2.625} = 22.8$$

From the spaced column formula, $\frac{P}{A} = 1000\left[1 - \frac{1}{3}\left(\frac{22.8}{40.5}\right)^4\right] =$ 965 lb. per sq. in.

$$\text{Required area} = \frac{43{,}400}{965} = 45 \text{ sq. in.}$$

Two 3 by 10 in. pieces with a cross-sectional area of 49.88 sq. in. will satisfy. Because of the short distance between the panel points, it may be found necessary to increase the width in order to accommodate the connectors in the top chord splice or in one of the joints.

Bottom chord:

Maximum load = 43,300 lb.

$$\text{Required area} = \frac{43{,}300}{1400} = 31 \text{ sq. in.}$$

Two 2 by 10 in. members with a cross-sectional area of 30.88 sq. in. will be sufficient. The width of these members may also have to be increased to accommodate the connectors in the bottom chord splice.

Web members:

The size of the web members will usually depend upon the joint design. If the thickness of the diagonal tension and vertical compression members is taken as $2\frac{5}{8}$ in., the placing of the connectors and the number of connectors in each joint will determine the width. This size should, however, be checked to make sure the members have ample cross-sectional area to carry their load.

Design of Splices

Top chord splice in U_4:

Split ring size, 4″
Number of contact faces for each ring, 2
Thickness of lumber, $2\frac{5}{8}$″
Angle of load to grain, 0°

Standard design load for one connector, 6,300 lb.
Load on joint, 41,500 lb.
Connectors required, 6.6
Connectors used, 8

Percentage of capacity developed, 82.5%

Reduced spacing, 7″

End distance standard, $5\frac{1}{2}$″

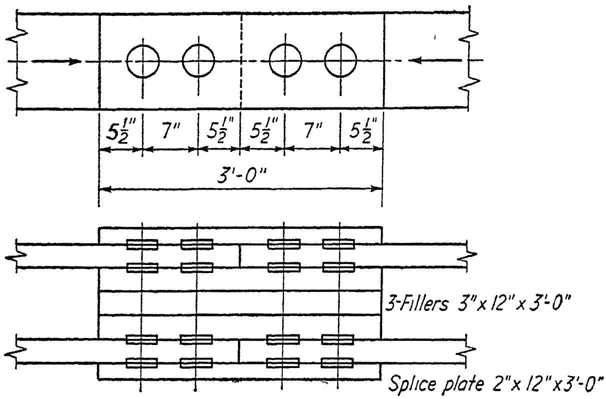


FIG. 65. Top chord splice in U_4.

Bottom chord splice in L_4:

Split ring size, 4″

Number of contact faces for each ring, 1

Thickness of lumber, $1\frac{5}{8}$″

Angle of load to grain, 0°

Standard design load for one connector, 6,400 lb.

Load on joint, 36,800 lb.

Connectors required, 5.75

Connectors used, 8

Percentage of capacity developed, 72%

Spacing standard, 9″

Spacing reduced (28%), 7″

Edge distance required, $2\frac{3}{4}$″

End distance standard, 7″

Reduced end distance (28%), $4\frac{1}{2}$″

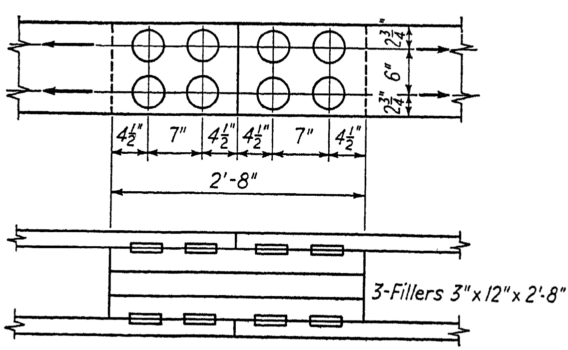


FIG. 66. Bottom chord splice in L_4.

Design of Joints

Top corner joint:

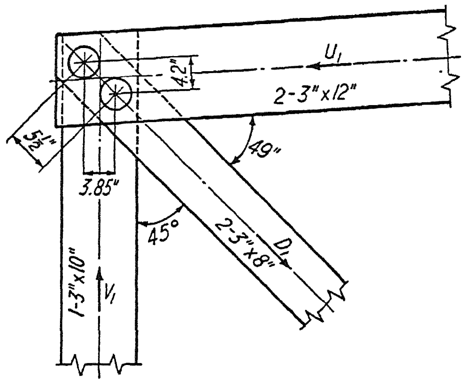


FIG. 67. Top corner joint for 60-ft. Pratt truss.

	V_1 to D_1	U_1 to D_1
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of lumber	$2\frac{5}{8}$″	$2\frac{5}{8}$″
Angle of load to grain	45°	49°
Standard design load for one connector	5,355 lb.	5,271 lb.
Load on joint	19,200 lb.	17,000 lb.
Connectors required	3.59	3.22
Connectors used	4	4
Percentage of capacity developed	90%	80%
Spacing parallel to grain in D_1	$5\frac{1}{2}$″	$5\frac{1}{2}$″
Edge distance, standard, compression side in D_1	$3\frac{3}{4}$″	$3\frac{3}{4}$″
Reduced edge distance	$3\frac{1}{4}$″	$2\frac{3}{4}$″

Width of members:

Diagonal, D_1, $3\frac{1}{4} + 2\frac{3}{4} = 6''$. Use $7\frac{1}{2}''$

Vertical, V_1, $2\frac{3}{4} + 3.85 + 2\frac{3}{4} = 9.35''$. Use $9\frac{1}{2}''$

Chord, U_1, $2\frac{3}{4} + 4.2 + 2\frac{3}{4} = 9.7''$. Use $11\frac{1}{2}''$

Standard design load for one connector at 0°	6,300 lb.	6,400 lb.

$$\text{Percentage of capacity developed, chord, } U_1 = \frac{17,000}{6400 \times 4} = 66.5\%$$

$$\text{Percentage of capacity developed, vertical, } V_1 = \frac{19,200}{6300 \times 4} = 76.2\%$$

Percentage of capacity developed, diagonal, $D_1 = \frac{23{,}900}{6300 \times 4} = 95\%$

Reduced end distance, chord, $U_1 = 3\frac{1}{2}''$
Reduced end distance, vertical, $V_1 = 4\frac{1}{4}''$
Reduced end distance, diagonal, $D_1 = 6\frac{1}{2}''$

Joint U_1–U_2:

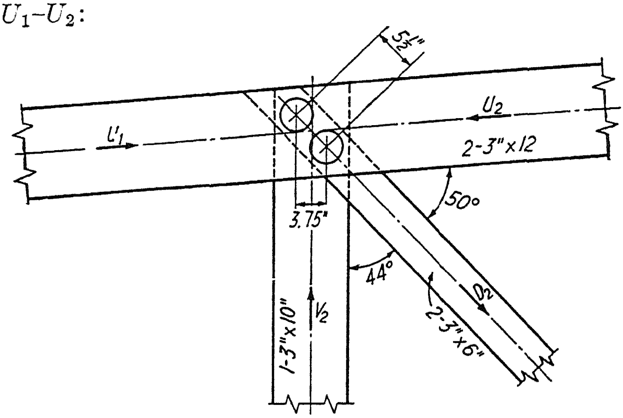


FIG. 68. Joint U_1-U_2.

	Chord to D_2	V_2 to D_2
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of lumber	$2\frac{5}{8}''$	$2\frac{5}{8}''$
Angle of load to grain	50°	44°
Standard design load for one connector	5,250 lb.	5,376 lb.
Load on joint	12,200 lb.	16,700 lb.
Connectors required	2.32	3.10
Connectors used	4	4
Percentage of capacity developed	58%	77.5%
Edge distance, standard, compression side in D_2	$3\frac{3}{4}''$	$3\frac{3}{4}''$
Edge distance used	$2\frac{3}{4}''$	$2\frac{3}{4}''$
Spacing parallel to grain in D_2	$5\frac{1}{2}''$	$5\frac{1}{2}''$

Width of members:

Diagonal, D_2, $2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}''$. Use $5\frac{1}{2}''$

Vertical, V_2, $2\frac{3}{4} + 3.75 + 2\frac{3}{4} = 9\frac{1}{4}''$. Use $9\frac{1}{2}''$

Standard design load for one connector at 0° 6,300 lb.

Percentage of capacity developed, vertical, $V_2 = \frac{16{,}700}{6300 \times 4} = 66.3\%$

Percentage of capacity developed, diagonal, $D_2 = \frac{17{,}700}{6300 \times 4} = 70.2\%$

Reduced end distance, vertical, $V_2 = 4\frac{1}{4}''$
Reduced end distance, diagonal, $D_2 = 4\frac{1}{4}''$

Joint U_2–U_3:

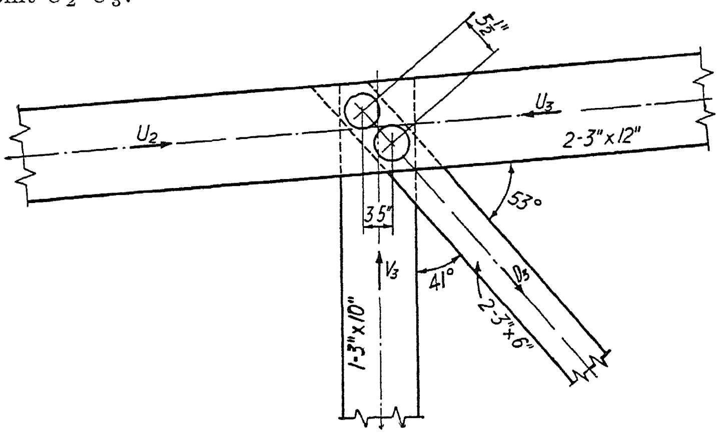


FIG. 69. Joint U_2–U_3.

	V_3 to D_3	Chord to D_3
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of lumber	$2\frac{5}{8}''$	$2\frac{5}{8}''$
Angle of load to grain	41°	53°
Standard design load for one connector	5,439 lb.	5,187 lb.
Load on joint	12,900 lb.	7,700 lb.
Connectors required	2.42	1.48
Connectors used	4	2
Percentage of capacity developed	60.5%	74%
Spacing parallel to grain in D_3	$5\frac{1}{2}''$	$5\frac{1}{2}''$
Edge distance, standard, compression side in D_3	$3\frac{3}{4}''$	$3\frac{3}{4}''$
Reduced edge distance	$2\frac{3}{4}''$	$2\frac{3}{4}''$

Width of members:

Diagonal, D_3, $2\frac{3}{4} + 2\frac{3}{4} = 5\frac{1}{2}''$. Use $5\frac{1}{2}''$

Vertical, V_3, $2\frac{3}{4} + 3.5 + 2\frac{3}{4} = 9''$. Use $9\frac{1}{2}''$

Standard design load for one connector at 0° 6,300 lb.

$$\text{Percentage of capacity developed, vertical, } V_3 = \frac{12{,}900}{6300 \times 4} = 51\%$$

$$\text{Percentage of capacity developed, diagonal, } D_3 = \frac{11{,}900}{6300 \times 4} = 47.3\%$$

Reduced end distance, vertical, $V_3 = 3\frac{1}{4}''$
Reduced end distance, diagonal, $D_3 = 3\frac{1}{2}''$

Joint L_1–L_2:

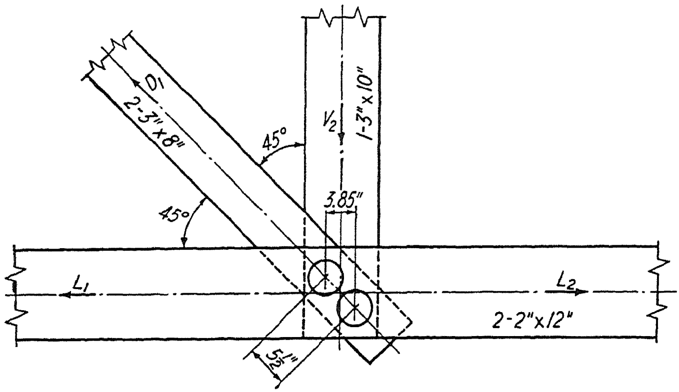


FIG. 70. Joint L_1–L_2.

	V_2 to D_1	Chord to D_1
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of lumber	$2\frac{5}{8}$″	$2\frac{5}{8}$″
Angle of load to grain	45°	45°
Standard design load for one connector	5,355 lb.	5,355 lb.
Load on joint	16,700 lb.	17,000 lb.
Connectors required	3.12	3.18
Connectors used	4	4
Percentage of capacity developed	78%	79.5%
Spacing parallel to grain in D_1	$5\frac{1}{2}$″	$5\frac{1}{2}$″
Edge distance, standard, compression side in D_1	$3\frac{3}{4}$″	$3\frac{3}{4}$″
Reduced edge distance:	$2\frac{3}{4}$″	$2\frac{3}{4}$″

It would be possible from this design to use a member $5\frac{1}{2}$″ wide for D_1. However, the design of the top corner joint requires that the width of D_1 be $7\frac{1}{2}$″.

Width of members:

Chord, L_1, $2\frac{3}{4} + 3.85 + 2\frac{3}{4} = 9.35$″. Use $11\frac{1}{2}$″

Vertical, V_2, $2\frac{3}{4} + 3.85 + 2\frac{3}{4} = 9.35$″. Use $9\frac{1}{2}$″

Diagonal, D_1, $3\frac{3}{4} + 3\frac{3}{4} = 7\frac{1}{2}$″. Use $7\frac{1}{2}$″

Standard design load for one connector at 0° 6,300 lb.

$$\text{Percentage of capacity developed, vertical, } V_2 = \frac{16{,}700}{6300 \times 4} = 66.5\%$$

$$\text{Percentage of capacity developed, diagonal, } D_1 = \frac{23{,}900}{6300 \times 4} = 95\%$$

Reduced end distance, vertical, $V_2 = 3\frac{1}{2}$″

Reduced end distance, diagonal, $D_1 = 6\frac{1}{2}$″

By placing a cross bolt between the ring and the end of the piece, the standard design loads may be used with end distances for tension members the same as those for compression members. In the previous joint, placing a cross bolt between the ring and the end of member D_1 makes it possible to use an end distance of $5\frac{1}{2}''$ in developing the full capacity of the rings. Since only 95% of the capacity of the rings is developed, the end distance may be reduced to $5\frac{1}{4}''$.

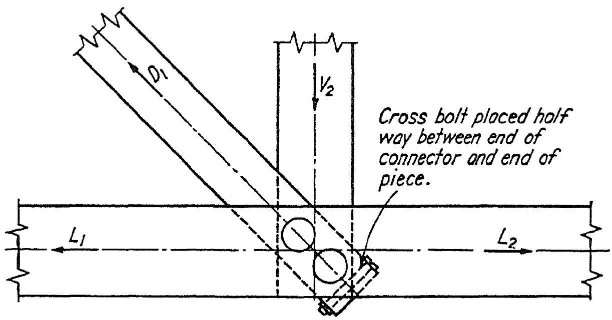


FIG. 71. Placement of cross bolts in tension members.

Joint L_2–L_3:

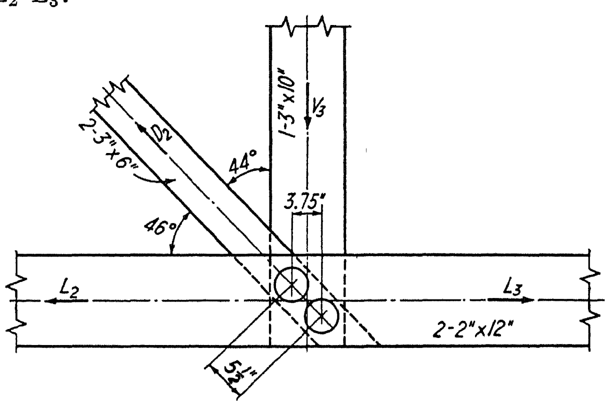


FIG. 72. Joint L_2–L_3.

	V_3 to D_2	Chord to D_2
Split ring size	4″	4″
Number of contact faces for each ring	2	2
Thickness of lumber	$2\frac{5}{8}''$	$2\frac{5}{8}''$
Angle of load to grain	44°	46°
Standard design load for one connector	5,376 lb.	5,334 lb.
Load on joint	12,900 lb.	12,100 lb.

	V_3 to D_2	Chord to D_2
Connectors required	2.4	2.27
Connectors used	4	4
Percentage of capacity developed	60%	57%
Spacing parallel to grain in D_2	$5\frac{1}{2}''$	$5\frac{1}{2}''$
Edge distance, standard, compression side in D_2	$3\frac{3}{4}''$	$3\frac{3}{4}''$
Reduced edge distance	$2\frac{3}{4}''$	$2\frac{3}{4}''$

Width of members:

Chord, L_2, $2\frac{3}{4} + 4 + 2\frac{3}{4} = 9.5''$. Use $11\frac{1}{2}''$

Vertical, V_3, $2\frac{3}{4} + 3.75 + 2\frac{3}{4} = 9.25''$. Use $9\frac{1}{2}''$

Diagonal, D_2, $2\frac{3}{4} + 2\frac{3}{4} = 5.5''$. Use $5\frac{1}{2}''$

Standard design load for one connector at 0° 6,300 lb.

$$\text{Percentage of capacity developed, vertical, } V_3 = \frac{12{,}900}{6300 \times 4} = 51.3\%$$

$$\text{Percentage of capacity developed, diagonal, } D_2 = \frac{17{,}700}{6300 \times 4} = 70.3\%$$

Reduced end distance, vertical, $V_3 = 3\frac{1}{4}''$

Reduced end distance, diagonal, $D_2 = 4\frac{1}{4}''$

71. Column and Wall Attachments. The method of anchoring trusses to wood columns can be best explained by means of Figure 73. It

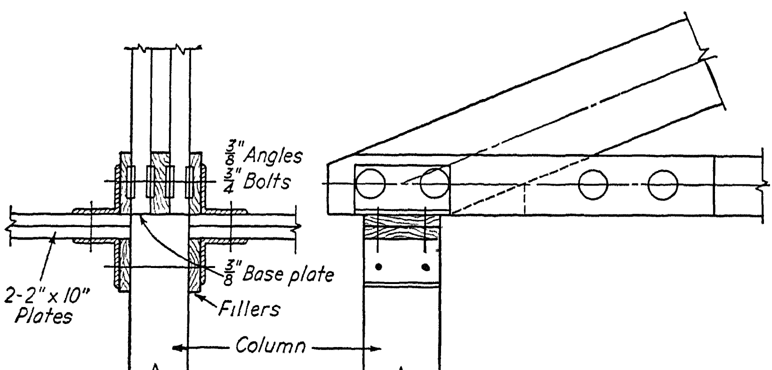


FIG. 73. Method of anchoring wood trusses to wood columns.

is often desirable to use a built-up column and have it act as an integral part of the truss. This type of construction is illustrated in Figure 74. Attaching a wood truss to a brick or concrete wall requires the use of anchor bolts that are fastened to the angles on the sides of the bottom chord.

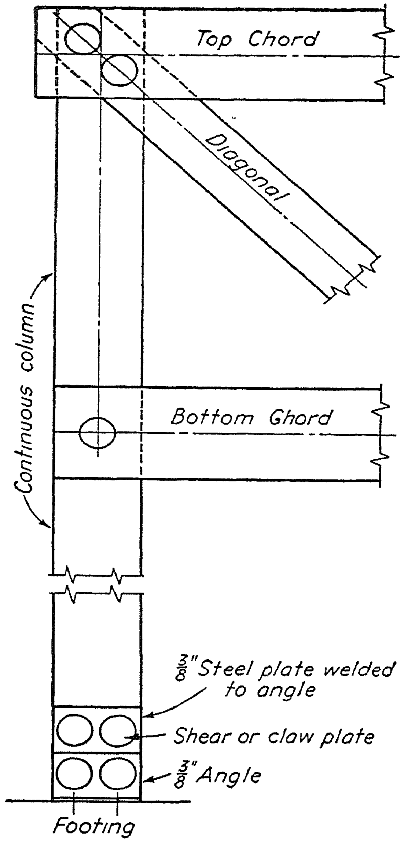


FIG. 74. Built-up columns made an integral part of the truss.

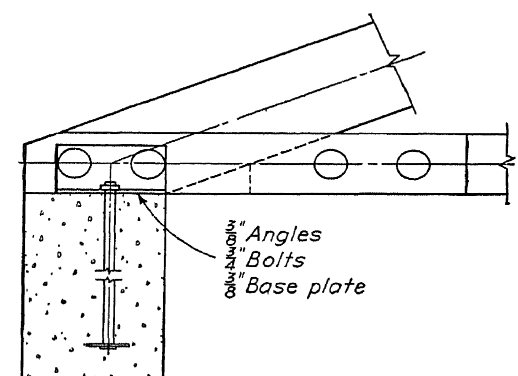


FIG. 75. Method of anchoring wood trusses to masonry walls.

72. Camber. Connectors have been used in the design of the trusses in this chapter, but the possibility of using bolts only in many joints or bolts in addition to the connectors should not be overlooked. Before connectors were used in this country it was common practice to design trusses with bolted joints, using steel rods for the tension members. This made it possible to put the camber in the truss after it was erected by merely tightening the tension rods. Moreover, wood will sag in time, and the above method of construction permitted the truss to be raised to the proper camber after several years of service. The usual cambers were between 2 and 4 in.

The steel rods at the center of the Fink trusses designed in this chapter have somewhat the same purpose of raising the bottom chord or at least in keeping it from sagging. However, in most trusses designed with connectors and all wood members it is necessary to fabricate and erect the truss with the camber already provided. The amount of camber depends upon the size of the truss, but a good estimate is between 1 in. and 6 in. for spans between 30 and 100 ft. The following table, which lists the total slip for joints with connectors, may be helpful in estimating the movement in the joints of wood trusses.

Table 26. Deformation, in Inches, for Joints Using Connectors

			Total Deformation	
Connector	Size	Bolt	*Load Parallel to Grain*	*Load Perpendicular to Grain*
Split rings	$2\frac{1}{2}$	$\frac{1}{2}$	0.040	0.050
	4	$\frac{3}{4}$	.045	.055
	6	$\frac{3}{4}$	.050	.060
Toothed rings	2	$\frac{1}{2}$	.004	.006
	$2\frac{5}{8}$	$\frac{5}{8}$	.006	.008
	$3\frac{3}{8}$	$\frac{3}{4}$	.008	.010
	4	$\frac{3}{4}$	.010	.012
Claw plates	$2\frac{5}{8}$	$\frac{1}{2}$	.015	.023
	$3\frac{1}{8}$	$\frac{1}{2}$	.038	.045
	4	$\frac{3}{4}$	.038	.045
Shear plates	$2\frac{5}{8}$	$\frac{3}{4}$	.073	.100
	4	$\frac{3}{4}$	.098	.102
	4	$\frac{7}{8}$	.050	.090
Spike grids, flat		$\frac{3}{4}$	.012	.013
		1	.015	.021
single curve		$\frac{3}{4}$	.011	.020
		1	.018	.025
double curve		$\frac{3}{4}$	.011	.020
		1	.018	.025

A formula developed by the Timber Engineering Company for calculating the camber in trusses built with connectors is as follows:

$$\Delta = K_1 \frac{L^3}{H} + K_2 \frac{L^2}{H}$$

in which Δ = recommended camber in inches at center of truss,
L = span of truss in feet,
H = height of truss in feet at center,
K_1 = 0.000032 for any type of truss,
K_2 = 0.0028 for flat and pitched trusses, or 0.00063 for bowstring trusses, i.e., trusses without splices in upper chord.

PROBLEMS

1. If the bottom chord of a Fink truss consists of two 3 by 8 in. pieces and is spliced at the heel joint with the top chord of two 3 by 14 in. pieces entering at an angle of 22°, how many connectors are required to carry a load of 43,000 lb. in the top chord? Sketch the joint showing the required spacing and the end and edge distances for the connectors.

2. The bottom chord of a truss consists of two 3 by 8 in. members in tension with loads on each side of a panel point of 34,300 lb. and 22,000 lb. Two diagonal 2 by 6 in. tension members enter the joint at 45° to the chord and carry a load of 11,500 lb. A single 3 by 6 in. compression member carrying a load of 8400 lb. enters the joint at 67½° to the chord. What size and how many connectors are required? Detail the joint.

3. Design and detail joint L_0–L_1, shown in Figure 52.

4. Design and detail the top and bottom chord splices for the truss illustrated in Figure 52.

5. For the truss shown in Figure 59, design and detail joint U_1–U_2.

6. For the same truss as in the previous problem, design and detail joint D_3–D_4.

7. Design and detail the peak joint for the Pratt truss shown in Figure 63.

8. Design and detail a 40-ft. Pratt truss with eight panels 5 ft. long. The end height of the truss is 4 ft. and the center height is 5 ft. 4 in. The trusses are spaced 16 ft. on centers. Consider the weight of the truss and assume the weight of purlins, sheathing, and roofing as 10 lb. per sq. ft. The snow load is 10 lb. per sq. ft. and the wind load is 15 lb. per sq. ft. Use select structural Douglas fir.

CHAPTER VI

TIMBER DECKS AND BRIDGES

73. General. Timber bridges may be divided into two general classes, namely, the trestle and the truss bridge. A trestle bridge usually consists of a number of pile bents, and the simplest form is to use the pile as both post and footing. However, in many trestle bridges the piles are driven and capped at the ground line and a frame bent is built on top of the pile foundation. In either type of construction it is necessary to provide bracing to keep the structure from swaying.

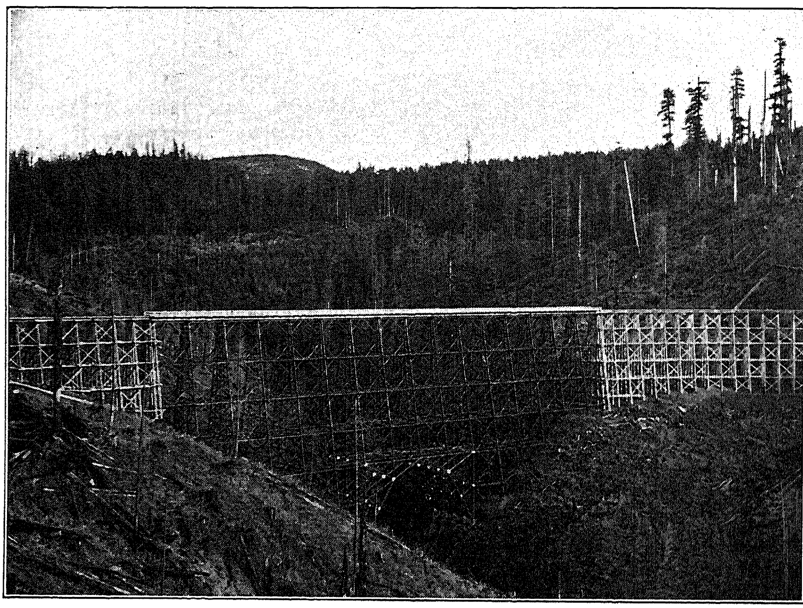


FIG. 76. Baird Creek Canyon Bridge constructed for Weyerhauser Timber Company. The bridge has an over-all length of 1130 ft. and a rise of 235 ft.

The truss bridge in its simplest form is the A-frame or King post truss and is suitable for spans up to about 40 ft. The Queen post truss is an adaptation of the A-frame with one panel of horizontal top chord added between the end posts. These have been used for spans up to 70 ft. A number of large timber truss bridges have been built recently with spans

of 150 to 210 ft. All these have utilized the connector system of construction, which provides economies and spans that heretofore have been impossible.

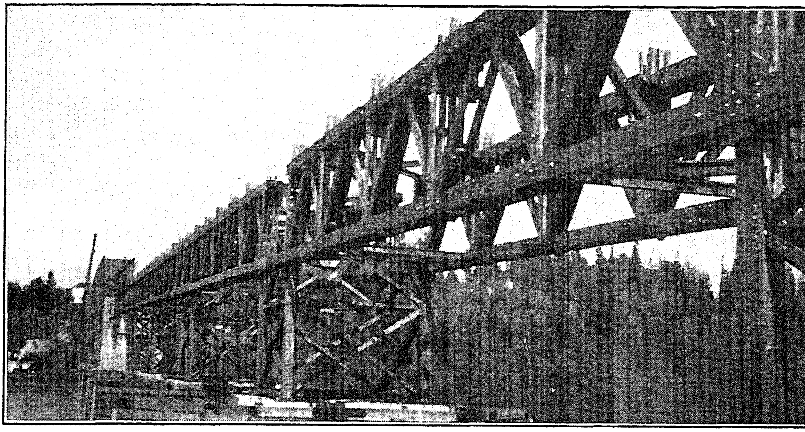


FIG. 77. Puget Island-Cathlamet, Washington, Bridge over Columbia River. Bridge is 2400 ft. long and is built with Wolmanized Douglas fir and Teco connectors. Fabricated by Timber Structures, Inc., and built by Washington Dept. of Highways.

In both types of bridges the lumber should be given a pressure treatment of an approved preservative in accordance with current practice

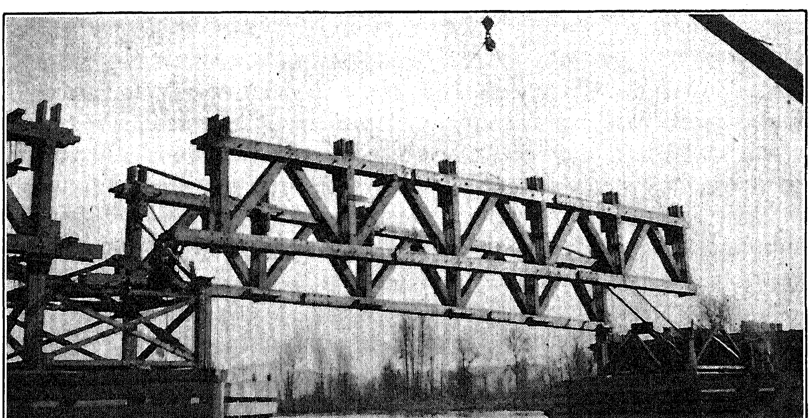


FIG. 78. A 100-ft. truss for the Puget Island-Cathlamet, Washington, Bridge being swung into place by a large crane on a barge.

for the condition of exposure of the structure. Moreover, the exposed portions of treated pieces that have been cut in the field should be given

an adequate treatment to make them as durable as the remaining portion. Piles that have been cut off are usually protected by several coats of hot creosote and pitch and a cap of galvanized iron. Pre-framing and boring of timbers at the treating plant, before the preservative is applied, is recommended. When it is necessary to bore a treated timber, the hole should be thoroughly soaked with hot preservative.

Bridge decks are usually made up of a stringer system with a plank or laminated floor or with continuous timber-concrete composite assem-

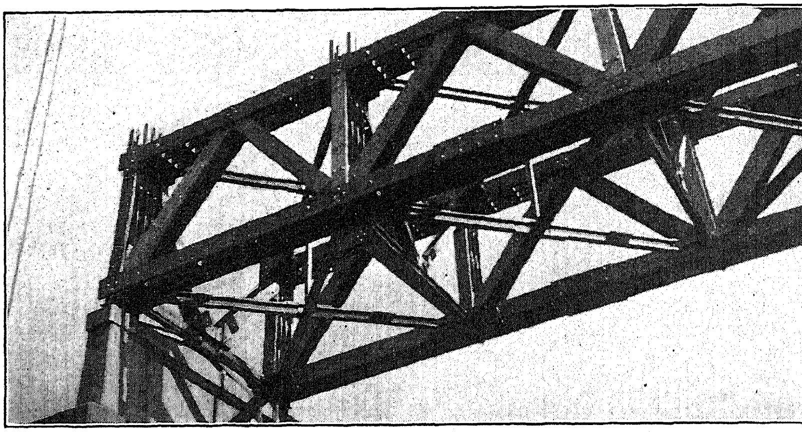


FIG. 79. Close up view of Wolmanized Douglas fir trusses for Puget Island-Cathlamet, Washington, Bridge.

blies. This latter form, which combines the ability of wood to resist tensile stresses and concrete to resist compressive stresses, has proved successful. The timbers are placed on edge with adjacent timbers varying in depth by about 2 in. This forms longitudinal grooves into which are driven metal plates to provide a shear connection between the timber on the bottom and the concrete mat on the top of the assembly.

74. Factors Affecting Design. The design of any structure depends upon three main factors: adequacy, economy, and aesthetics. By adequacy is meant that the structure should be so designed that it will resist the loads and forces to which it will be subjected with a reasonable factor of safety. Moreover, the structure should be adequate to perform the function for which it is intended. A bridge is part of a railway or highway system and, as such, should be designed in relation to the primary project of which it is probably only a small part. The existence of numerous timber trestles in railroad construction and truss and deck bridges along our highways seems to be proof that timber bridges can be

designed to adequately carry the loads ordinarily encountered in this type of construction.

The economy of any design depends upon the initial cost, maintenance cost, and permanency of the structure. The relative economy of different types of structures is not determined by the initial costs alone but rather by comparing the annual cost which includes a portion of the initial cost, maintenance charges, and an annuity which will produce a sum sufficient to reproduce the structure. The items of first cost and interest charges are known quantities or can be estimated for any type of bridge. Consequently, maintenance costs and the permanency of a structure are the two considerations involved in rating one design more advantageous than another. Maintenance cost depends upon the durability of the material. Statistics are available which show that the average life of creosoted railway crossties is not less than 30 years, that exposed creosoted timber piles have an average life of 35 to 40 years, and stringers and decking in ballasted deck bridges, 40 years. In the ballasted deck bridges used in railway structures the deck material is protected from exposure and so is not subject to mechanical wear. This type of structure may be considered equivalent to highway bridges having a suitable wearing surface on top of the timber deck.

The maintenance cost for any structure is not readily estimated without actual service records and costs of similar structures. Timber decks with a suitable wearing surface usually require maintenance of the wearing surface before it is necessary to repair or replace any of the beams or stringers. The timber-concrete composite deck has the best rating from a maintenance standpoint. Table 27 gives the total amount expended for maintenance and repairs for each year of service for 129 ballasted deck creosoted bridges. From the figures it is possible to calculate the cost per linear foot per year of age.

An estimate of the permanence of any structure requires consideration of the fact that the structure may become obsolete or inadequate before the end of its expected life. Consequently, in estimating the life of a structure the factor of permanence should be considered with caution. While it is true that the expected life may be measured in centuries, a study of the histories of many bridges will show obsolescence from anywhere between 10 and 50 years.

Many timber bridges are not very beautiful structures, but there seems to be no reason why properly designed timber bridges cannot be made as pleasing in appearance as other structures. One of the main requirements of aesthetics in design is to produce a structure that will be in harmony with its surroundings. Proper treatment of balustrades, and the increased use of built-up members and composite decks can add much to the appearance of timber bridges.

Table 27. Maintenance and Repair Costs for 129 Ballasted Deck Creosoted Timber Railway Bridges in Louisiana

["Timber Bridge Structures," by J. F. Seiler, *Am. Road Builders' Assoc.*, Bull. 43-A, 1935. From Office of Chief of Engineers, U. S. Army]

Total Costs for Year of Service Shown

Bridge Number	1	2	3	4	5	6	7	8
1 to 20 Inc	$22 09	$101 66	$ 27 56	$ 48 33	$103 69	$232 11	$292 45	$ 80.56
21 to 43 Inc.	35 98	58 54	177 98	284 17	405 53	38 55	769 36	1,311.14
44 to 64 Inc.	....	136 19	41.52	133 27	403 46	170 17	488 75	191.53
65 to 85 Inc.	....	45 06	....	156 79	107 00	76 80	184 63	297 78
86 to 107 Inc	....	....	....	....	95.34	....	740 74	131 62
108 to 129 Inc.	....	....	....	....	205 15	....	....	....
Totals	$58 07	$341 45	$247.06	$622.56	$1,320 17	$517 63	$2,475 93	$2,012 63
Footage in Service	15,571	15,571	15,571	15,455	15,455	15,252	15,162	15,121

Bridge Number	9	10	11	12	13	14	15	16
1 to 20 Inc	$1,100.22	$ 17.57	$ 37.38	$ 844 13	$ 868 11	$403 82	$1,005 73	$ 301.16
21 to 43 Inc.	242.98	4,393.72	1,450 76	1,299 25	1,365.24	542 88	410.95	1,945.40
44 to 64 Inc.	131.69	125 48	40.55	162 43	36.53	517 22	48 34	163.48
65 to 85 Inc	99 86	185 82	65.70	46 98	205 18	310 31	211.11	229.70
86 to 107 Inc	215 93	44 04	432 14	30 79	230 71	305 65	129.18	313 97
108 to 129 Inc.	....	552 52	429 46	83 45	....	367 20	62.46	....
Totals	$1,790 68	$5,319.15	$2,455 59	$2,467.03	$2,705.77	$2,447.08	$1,867.77	$2,953.71
Footage in Service	15,058	14,408	14,381	12,189	11,619	10,406	9,368	6,894

Bridge Number	17	18	19	20	21	22	23	24
1 to 20 Inc.	$ 240 84	$ 622.44	$131 92	$278.58	$1,577.38	$ 98 48	$63.46	....
21 to 43 Inc.	988 49	333.94	87 57	44 68	48 45	106.32	77.85	....
44 to 64 Inc.	285 92	169.80	48.09	49.53	174.79	74.02	....	....
65 to 85 Inc.	232 06	372 28	....	....	330.26	....	....	....
86 to 107 Inc.	537.99	91.88	....	....	104.29	....	....	....
108 to 129 Inc	....	....	....	....	....	....	....	....
Totals	$2,285.30	$1,590 34	$267.58	$372 79	$2,235.17	$278.82	$141.31	....
Footage in Service	5,892	4,005	1,698	996	914	805	709	615

A.A.S.H.O. SPECIFICATIONS

The American Association of State Highway Officials has set forth specifications for the selection of material and the design of highway bridges in its Standard Specifications for Highway Bridges. The following articles are taken from or based on these specifications.

75. Timber Piles. Timber piles which will be below water level at all times may be of any species of wood which will satisfactorily withstand driving.

In untreated piling for use in exposed work, the diameter of the heartwood shall be not less than eight-tenths of the required diameter of the pile.

All wood piling shall be cut from sound and live trees, preferably during the winter season. They shall contain no unsound knots. Sound knots will be permitted, provided the diameter of the knot does not exceed 4 in. or one-third of the diameter of the stick at the point where it occurs. Any defect or combination of defects which will impair the strength of the pile more than the maximum allowable knot shall not be permitted. The butts shall be sawed square, and the tips shall be sawed square or tapered to a point not less than 4 in. in diameter as directed by the engineer.

Unless otherwise specified, all piles shall be peeled by removing all of the rough bark and at least 80 per cent of the inner bark. No strip of inner bark remaining on the stick shall be over ¾ in. wide or over 8 in. long, and there shall be at least 1 in. of clean wood surface between any two such strips. Not less than 80 per cent of the surface of any circumference shall be clean wood.

Piles shall be cut above the ground swell and shall taper from butt to tip. A line drawn from the center of the tip to the center of the butt shall not fall outside of the center of the pile at any point more than 1 per cent of the length of the pile. In short bends, the distance from the center of the pile to a line stretched from the center of the pile above the bend to the center of the pile below the bend shall not exceed 4 per cent of the length of the bend or a maximum of 2½ in. All knots shall be trimmed close to the body of the pile.

Round piles shall have a minimum diameter at the tip, measured under the bark as follows:

Length of Pile	*Tip Diameter*
Less than 40 ft.	8 in.
40 to 60 ft.	7 in.
Over 60 ft.	6 in.

The minimum diameter of piles at a section 3 ft. from the butt, measured under the bark, shall be as follows:

	Diameter in Inches	
Length of Pile	*Douglas Fir, Southern Yellow Pine*	*All Other Species*
20 ft. and under	11	11
21 to 30 ft.	12	12
31 to 40 ft.	12	13
Over 40 ft.	13	14

The diameter of the pile at the butt shall not exceed 20 in. The diameter of a pile in cases where the tree is not exactly round shall be determined either by measuring the circumference and dividing the number of inches by 3.14, or by taking the average of the maximum and minimum diameters at the location specified.

When required, the size and number of piles shall be determined by actual loading tests. In general, these tests shall consist of the application of a test load placed upon a suitable platform supported by the pile, with suitable apparatus for accurately measuring the test load and the settlement of the pile under each increment of load.

In lieu thereof hydraulic jacks with suitable yokes and pressure gages may be used.

The safe allowable load shall be considered as 50 per cent of that load which, after a continuous application of 48 hr., produces a permanent settlement not greater than ¼ in. measured at the top of the pile. This maximum settlement shall not be increased by a continuous application of the test load for a period of 60 hr. or longer. At least one pile for each group of 100 piles shall be tested.

In the absence of loading tests, the safe bearing values for timber piles shall be determined by the following formulas:

$$P = \frac{2WH}{S + 1.0} \text{ for gravity hammers}$$

$$P = \frac{2WH}{S + 0.1} \text{ for single-acting steam hammers}$$

$$P = \frac{2H(W + Ap)}{S + 0.1} \text{ for double-acting steam hammers}$$

where P = safe bearing power, in pounds,
W = weight, in pounds, of striking parts of hammer,
H = height of fall, in feet,
A = area of piston, in square inches,
p = steam pressure in pounds per square inch at the hammer,
S = the average penetration in inches per blow for the last 5 to 10 blows for gravity hammers and the last 10 to 20 blows for steam hammers.

The above formulas are applicable only when
(*a*) the hammer has a free fall,
(*b*) the head of the pile is not broomed or crushed,
(*c*) the penetration is reasonably quick and uniform,
(*d*) there is no sensible bounce after the blow,
(*e*) a follower is not used.

Twice the height of the bounce shall be deducted from H to determine its value in the formula.

Unless otherwise ordered by the engineer, timber piling shall be driven to the bearing value given on the plans or in the supplemental specifications. If bear-

ing values are not given, timber piling shall be driven to a minimum value of 20 tons.

In case water jets are used in connection with the driving, the bearing power shall be determined by the above formulas from the results of driving after the jets have been withdrawn, or a load test may be applied.

76. Structural Timber. The stress grades for structural timber conform to the principles of strength-grading of the American Lumber Standards, Simplified Practice Recommendations, R-16, which represent complete grading rules suitable for use both in engineering design and lumber purchase and incorporate all factors affecting the strength and utility of structural timbers and their commercial grading. Existing commercial grading rules of the regional lumber manufacturers' associations are in conformity, generally, with the principles of structural grading set forth by the A.A.S.H.O. and provide for grades of equal or higher working stresses.

77. Timber Grades for Particular Uses. For the various structural purposes the grades given in Table 28 may be used.

78. Distribution of Loads. For calculating the bending moments in longitudinal beams or stringers, a lateral but no longitudinal distribution of wheel load is permitted. The proportion of the wheel load for calculating the moments in interior stringers may be determined as follows:

$$W' = \frac{WS}{K} \text{ for one traffic lane.}$$

$$W' = \frac{WS}{K'} \text{ for two traffic lanes.}$$

W' = fraction of wheel load to each stringer.
W = wheel load in pounds.
S = average spacing of stringers in feet.
K = 4.0 plank floors.
4.5 strip floor 4 in. thick.
5.0 strip floor 6 in. thick or thicker.
K' = 3.75 plank floors.
4.00 strip floor 4 in. thick.
4.25 strip floor 6 in. thick or thicker.

The live load supported by outside stringers is the reaction of the truck wheels, assuming the flooring to act as a simple beam between stringers.

TIMBER TRESTLES

79. General. The decks of timber trestles are usually constructed of 2 by 4 in. or 2 by 6 in. material laid on edge with their length perpen-

Table 28. Structural Grades for Highway Bridge Construction.

[Standard Specifications for Highway Bridges, A.A.S.H.O., 1941]

	Structural Purpose	Size of Member	Standard Grade [1]
(*a*)	Truss members, tension Floor beams Stringers Other floor members	5″ x 8″ and larger	1800#f, or 1600#f, or 1400#f structural beams and stringers
(*b*)	Caps Posts, bridge and guard rail Sills Mud sills Nailing strips Truss members, compression Timbers (culverts)	6″ x 6″ and larger	1200#c or 1100#c structural posts and timbers
(*c*)	Joist Decking, wearing Other floor members Rails Rail posts Nailing strips Truss members, compression and tension Guard rail	4″ and thinner	1800#f, 1600#f, or 1400#f structural joist and plank
(*d*)	Wheel and felloe guards	6″ x 6″ and larger	1100#c structural posts and timbers
(*e*)	Sub-decking, flat Sub-decking, laminated Bracing, sway, sash, and longitudinal Girts Bulkhead plank Scupper blocks Cleats Grillage	4″ and thinner	1200#f or 1100#f joist and plank
(*f*)	Cross-bridging Sidewalk Firestops	2″ and 3″ thick	No. 1 dimension
(*g*)	Truss housing Inside sheathing	1″ and $1\frac{1}{4}$″ thick	D select finish, or No. 1 boards

[1] For temporary structures which are for use only during erection or for emergency use, the grades of 1200#f or 1100#c may be substituted for 1800#f, 1600#f, 1400#f, or 1200#c where specified above; No. 1 Dimension for 1200#f and 1100#f, and No. 1 timbers for 1100#f.

dicular to the direction of the traffic. This type of deck distributes the load over a greater area than the plank deck where the individual pieces are laid flat. Moreover, the edge loading conditions are less severe in the laminated deck because the pieces can be effectively tied together to act as one solid unit.

Span lengths up to 25 ft. are economical for timber trestles where all wood construction is used. However, beyond 25 ft. it is usually more economical to use rolled steel beams for the stringer system. On span lengths over 15 ft. where wood stringers are used it is customary to bolt 4 by 12 in. blocks between the stringers to act as bridging.

80. Loads. Bridges on main arteries of traffic are designed for H-15 or H-20 loading and on less important routes for H-10 loading in accordance with Standard Specifications for Highway Bridges of the A.A.S.H.O. These loading conditions indicate trucks weighing 10, 15, and 20 tons, with each rear wheel carrying 0.4 of the total load and each front wheel carrying 0.1 of the load.

In a laminated deck, where the individual pieces extend across several stringers, the floor may be designed on the basis of 0.8 of the bending moment for a simple beam. The wheel load distribution is assumed to be 15 in. in a direction normal to the laminations. For plank floors the wheel load distribution is assumed as the width of the individual plank unless the planks are more than 5½ in. thick, tongue and grooved, or splined together. In this case the wheel load distribution may be taken as four times the thickness.

81. Stringer Design. Assume a timber trestle with the distance between the centers of the bents as 20 ft., and design the stringers for an H-15 loading. The roadway width is 24 ft. and the timber stringers and deck have a working stress in tension of 1400 lb. per sq. in. and weigh 50 lb. per cu. ft. The weight of the wearing surface is taken as 25 lb. per sq. ft.

EXAMPLE. Assume 12 rows of 8 by 16 in. stringers.

$$\text{Spacing center-to-center} = \frac{23.3}{11} = 2.12 \text{ ft.}$$

Dead load:

$$\text{Stringer} = \frac{8 \times 16}{144} \times 20 \times 50 = 890 \text{ lb.}$$

$$\text{4-in. deck} = \frac{4 \times 2.12 \times 20 \times 50}{12} = 707 \text{ lb.}$$

$$\text{Wearing surface} = 2.12 \times 20 \times 25 = \underline{1060 \text{ lb.}}$$

$$\text{Total} = 2657 \text{ lb.}$$

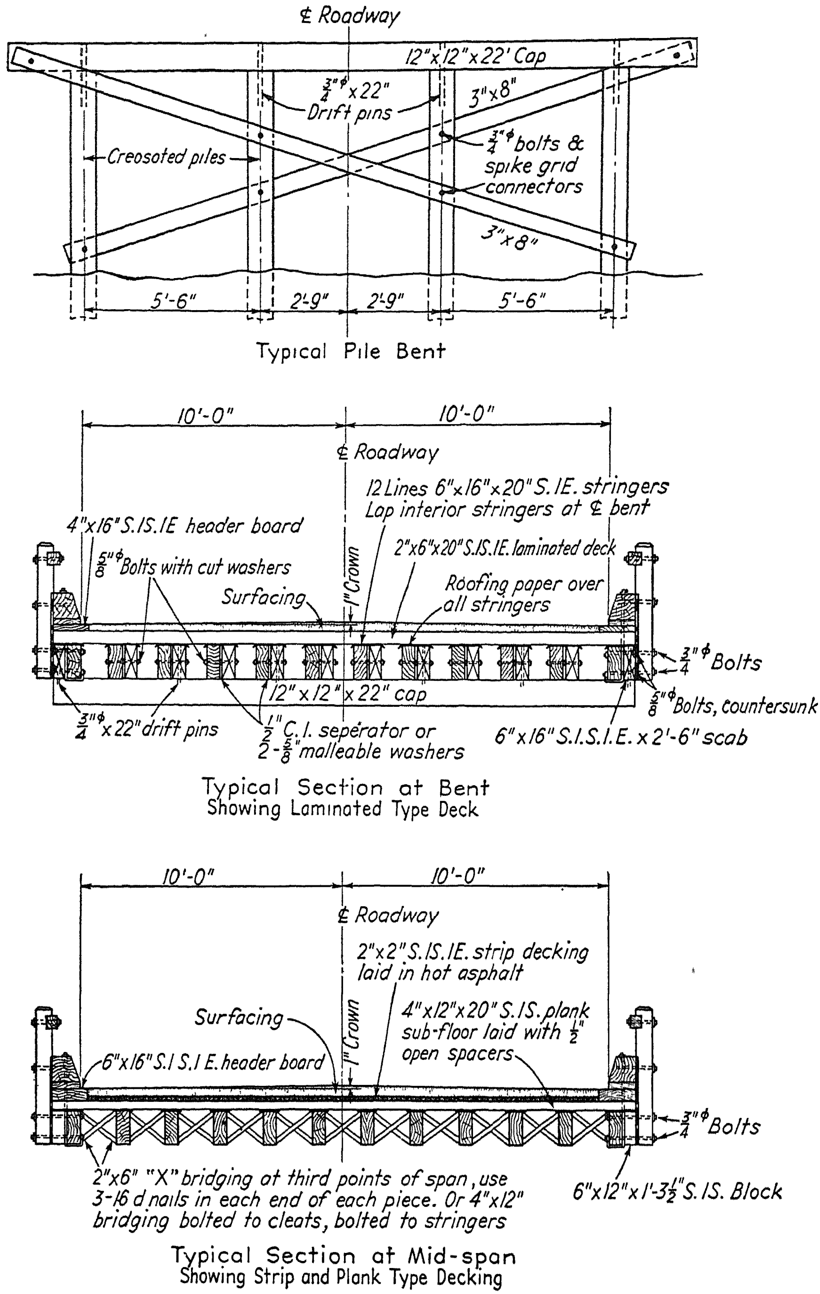


FIG. 80. Timber trestle details.

Live load:

H-15 loading, rear wheel weighs 12,000 lb.

$$W' = \frac{WS}{K'} = \frac{12{,}000 \times 2.12}{4} = 6360 \text{ lb.}$$

Moments:

$$\text{Dead load moment} = \frac{2657 \times 20 \times 12}{8} = 79{,}710 \text{ in.-lb.}$$

$$\text{Live load moment} = \frac{6360 \times 20 \times 12}{4} = 381{,}600 \text{ in.-lb.}$$

$$\text{Total} = 461{,}310 \text{ in.-lb.}$$

Fiber stresses:

Allowable working stress = 1400 lb. per sq. in.

$$\text{Actual unit stress} = \frac{6M}{bd^2} = \frac{6 \times 461{,}310}{8 \times 16 \times 16} = 1350 \text{ lb. per sq. in.}$$

Shear:

In computing the horizontal shear, place the wheel load at a point three times the depth of the beam from the support, which may be taken at the quarter point. It may be assumed that the stringer is relieved of 80 per cent of the portion of the load that is carried by the other stringers when the load is at the midspan.

Portion of load carried by other stringers when load is at midspan = 12,000 − 6360 = 5640 lb.

Load acting on stringer at quarter point = 12,000 − 0.8 × 5640 = 7490 lb.

In determining the shear due to dead load the uniform load from the end of the stringer to a point equal to the height of the beam should be disregarded.

Total shear:

$$\text{Live load} = 7490 \times \tfrac{3}{4} = 5620 \text{ lb.}$$

$$\text{Dead load} = \frac{2657}{2}\left(\frac{10 - 1.3}{10}\right) = 1154 \text{ lb.}$$

$$\text{Total} = 6774 \text{ lb.}$$

Unit shear:

Allowable = 100 lb. per sq. in.

$$\text{Actual} = \frac{3}{2}\,\frac{6774}{8 \times 16} = 79.3 \text{ lb. per sq. in.}$$

TIMBER-CONCRETE COMPOSITE DECKS

82. General. The American Wood Preservers' Association is the owner of the patent covering treated timber-concrete composite decks, but it is the policy of the Association to make this type of construction available to the public without undue restrictions.

The treated timber-concrete composite deck is a slab consisting of a laminated timber base that is interlocked with a concrete mat. The concrete mat performs two functions: it helps support the loads and acts as an extremely durable wearing surface. The treated timber base is made up of 2-in. planks laid on edge with varied widths of adjacent planks. The usual method is to use a 2 by 4 in. piece with a 2 by 6 in. piece next to it, alternating the widths across the width of the roadway. Metal plates called "shear developers" are driven into the longitudinal

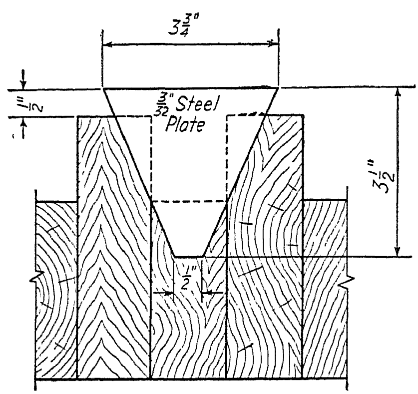


FIG. 81. Shear developers.

grooves with uplift spikes driven into the raised lamination at intervals of 2 to 4 ft.

The concrete mat is usually reinforced for shrinkage and temperature stresses to prevent cracking; this reinforcement consists of ⅜-in. or ½-in. round bars placed on 9 to 12 in. centers in both directions. When the deck is continuous over a support additional reinforcing is usually added to take care of the negative moments. Recently, however, a number of composite decks have been designed and built without any reinforcing, and they have not developed any evidence of weakness or cracks of a serious nature.

For single-span lengths all the planks should be full length, extending from support to support; but in multiple-span construction, one-third of the laminations should butt joint over the support and one-third at each quarter point. With this arrangement two-thirds of the laminations are effective at the support and the quarter points and a full section extends throughout the distance between these points.

83. Loads. In the construction of timber-concrete composite decks the treated timber is assembled in place first and the shear developers and uplift spikes driven before the concrete mat is poured. The timber

sub-deck carries all the dead load, so the first procedure in design is to determine the dead load stresses in the timber sub-deck.

Tests have shown that in continuous spans with the laminations arranged as previously described the positive and negative bending moments for uniform dead load in interior spans are very nearly equal and for practical purposes may be taken as 50 per cent of the simple span dead load moment.[1] The positive moments in the end spans may be assumed at 60 per cent of the simple span moment.

Results of tests by the Maryland State Roads Commission make it possible to consider a critical concentrated live load as laterally distributed over 5 ft. in calculating moment. For shear the distribution may be taken as 4 ft.

84. Design Assumptions. It is assumed that the ratio of the modulus of elasticity of concrete, E_c, and that of wood, E_w, is equal to one, and that the ratio of the modulus of elasticity of steel, E_s, to that of southern pine or Douglas fir is 18.75. The lateral distribution for concentrated loads in calculating moment is 5 ft. and for shear is 4 ft. The distribution of the dead load bending moments in continuous spans is taken as 50 per cent of the simple span dead load moment in interior spans and as 60 per cent in end spans. The distribution of the live load bending moments is taken as 70 per cent of the simple span live load moment for positive moments in interior spans and 50 per cent for negative moments. For end spans 85 per cent of the simple span moment is taken for positive moments and 40 per cent for negative moments.

85. Design. Assume a continuous composite deck as part of a highway bridge to carry an H-15 loading. The center lines of the bents are 20 ft. apart. Design the interior span of the composite deck, using a 1-ft. width in the calculations.

Working stresses:

Timber: 1600 lb. per sq. in., tension.
Concrete: 800 lb. per sq. in., compression.
Steel: 18,000 lb. per sq. in.
Shear developers: 1750 lb. each for steel $\frac{3}{32}$ in. thick in grooves 2 in. wide or less.

Weights:

Timber: 60 lb. per cu. ft.
Concrete: 150 lb. per cu. ft.

[1] Timber-Concrete Composite Decks, Service Bureau, American Wood Preservers' Association, April, 1941.

Wood section at midspan:

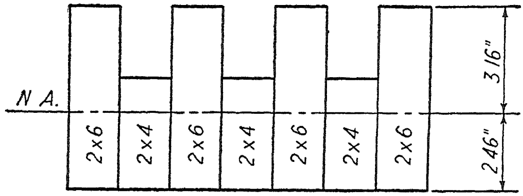


FIG. 82. Wood section at midspan.

$$6 \times 5.625 = 33.75 \qquad 33.75 \times 2.812 = 95.0 \qquad \frac{bd^3}{12} = \frac{6 \times 5.625^3}{12} = 88.90$$

$$6 \times 3.625 = \underline{21.75} \qquad 21.75 \times 1.812 = \underline{41.3} \qquad \frac{6 \times 3.625^3}{12} = 23.85$$

$$55.50 \qquad)136.3$$

$$y = 2.46 \qquad Ad^2 = 33.75 \times 0.35^2 = 4.13$$

$$21.75 \times 0.65^2 = \underline{9.18}$$

$$I_1 = 126.06$$

Wood section at support:

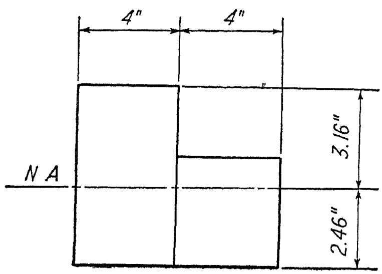


FIG. 83. Wood section at support.

$$I_2 = \tfrac{2}{3} \times 126.06 = 84.1$$

Composite section at midspan:

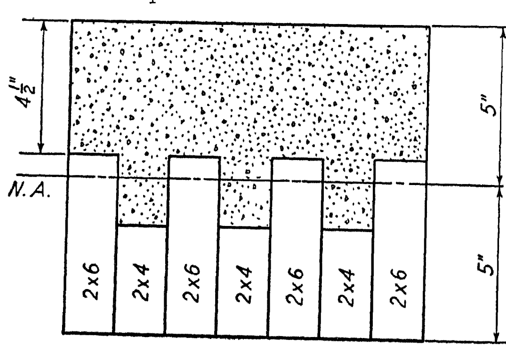


FIG. 84. Composite section at midspan.

This section is considered homogeneous.

$$y = 5''$$

$$I_3 = \frac{bd^3}{12} = \frac{12 \times 10^3}{12} = 1000$$

Composite section at support:

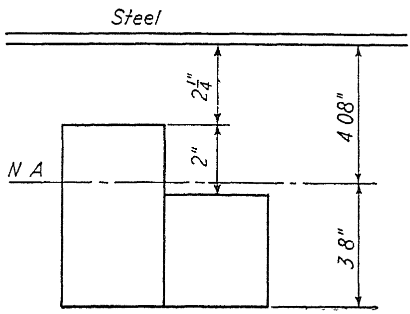


FIG. 85. Composite section at support.

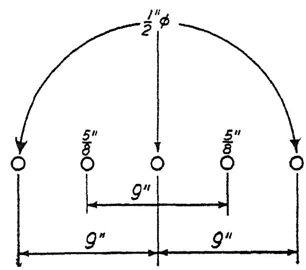


FIG. 86. Arrangement of steel.

$$\begin{aligned}
\tfrac{2}{3} \times 6 \times 5.625 &= 22.5 \times 5.06 = 114.0 \\
\tfrac{2}{3} \times 6 \times 3.625 &= 14.5 \times 6.06 = 87.8 \\
& \underline{12.56} \times 0 = \underline{0} \\
& 49.56 \qquad\quad)201.8
\end{aligned}$$

$$y_s = 4.08$$

$$y_w = 3.80$$

Steel: $\frac{5}{8}$ in. $\phi = 0.307 \times 2 = 0.614$

$\frac{1}{2}$ in. $\phi = 0.196 \times 2 = \underline{0.392}$

1.00

$1.00 \times \frac{12}{18} = 0.67$ sq. in. per ft. of slab width

$0.67 \times 18.75 = 12.56$ sq. in. of wood

$$\frac{bd^3}{12} = \frac{4 \times 5.625^3}{12} = 59.3$$

$$\frac{4 \times 3.625^3}{12} = 15.9$$

$$\begin{aligned}
Ad^2 = 22.5 \times 1^2 &= 22.5 \\
14.5 \times 2^2 &= 58.0 \\
12.56 \times 4.08^2 &= \underline{209.0} \\
I_4 &= 364.7
\end{aligned}$$

Dead loads and moments:

Timber dead load $= \dfrac{4.625}{12} \times 60 = 23.125$ lb. per sq. ft.

Concrete dead load $= \dfrac{5.5}{12} \times 150 = \underline{68.5}$ lb. per sq. ft.

91.625 lb. per sq. ft.

Simple span dead load moment $= \dfrac{91.625 \times 20 \times 20 \times 12}{8} = 55{,}000$ in.-lb.

Distribution of moment in interior spans:

Positive moment $= 55{,}000 \times 0.5 = 27{,}500$ in.-lb.

Negative moment $= 55{,}000 \times 0.5 = 27{,}500$ in.-lb.

Live loads and moments:

Wheel load $= 12{,}000$ lb.

Load per foot of width $= \dfrac{12{,}000}{5} = 2400$ lb.

Simple span moment $= \dfrac{2400 \times 20 \times 12}{4} = 144{,}000$ in.-lb.

Positive moment (interior span) $= 144{,}000 \times 0.7 = 100{,}800$ in.-lb.

Negative moment (interior support) $= 144{,}000 \times 0.5 = 72{,}000$ in.-lb.

Dead load stresses:

Tension at midspan, $f_w = \dfrac{27{,}500 \times 2.46}{126.06} = 537$ lb. per sq. in.

Compression at support, $f_w = \dfrac{27{,}500 \times 2.46}{84.1} = 805$ lb. per sq. in.

Live load stresses:

Midspan:

Tension in wood, $f_w = \dfrac{100{,}800 \times 5}{1000} = 504$ lb. per sq. in.

Compression in concrete, $f_c = \dfrac{100{,}800 \times 5}{1000} \times 1.3 = 655$ lb. per sq. in. where 30% is added for impact.

Support:

Compression in wood, $f_w = \dfrac{72{,}000 \times 3.8}{364.7} = 750$ lb. per sq. in.

Tension in steel, $f_s = \dfrac{72{,}000 \times 4.08}{364.7} \times 18.75 \times 1.3 = 19{,}650$ lb. per sq. in.

Maximum combined stresses:

Timber:

Midspan $= 537 + 504 = 1041$ lb. per sq. in.

Support $= 805 + 750 = 1555$ lb. per sq. in.

Concrete:

Midspan $= 655$ lb. per sq. in.

Steel:

Support $= 19{,}650$ lb. per sq. in.

The stress in the steel exceeds the allowable stress, so it will be necessary to increase the amount of steel over the support or to use a larger wood section.

Spacing of shear developers:

Lateral load distribution = 4 ft.

Critical position of load = 3 × depth of slab from support. This point is near timber joints and net section only is considered.

$$\text{Effective load per foot of width} = \frac{12{,}000}{4} = 3000 \text{ lb.}$$

Increase for impact = 3000 × 1.3 = 3900 lb.

$$\text{End reaction} = 3900 \times \frac{17}{20} = 3315 \text{ lb.}$$

Net section = (10 × 12) − (2 × 5.625) − (2 × 3.625) = 100.5 sq. in.

$$\text{Average unit shear} = \frac{3315}{100.5} = 33 \text{ lb. per sq. in.}$$

Maximum shear at neutral axis = 33 × 1.5 = 49.5 lb. per sq. in.
Total horizontal shear on 1 sq. ft. = 49.5 × 144 = 7130 lb.

$$\text{Shear developers required per square foot of contact area} = \frac{7130}{1750} = 4.07.$$

$$\text{Number of grooves per foot of width} = \frac{6}{1.625} = 3.69.$$

$$\text{Shear developers required per linear foot in each groove} = \frac{4.07}{3.69} = 1.1.$$

Spacing in each groove = 10.9 in. Use 10.5 in. This spacing may be increased as the shear decreases along the span.

PROBLEMS

1. Design a plank floor and stringer system for a timber trestle with bents 15 ft. on centers to carry an H-10 loading on an 18 ft. roadway. Use select structural Douglas fir.

2. Design an A-frame truss to span 20 ft. and carry an H-10 loading on a roadway 20 ft. wide. Use select structural Douglas fir.

3. For a span of 50 ft. and roadway 24 ft. wide design and detail a truss bridge to carry an H-15 loading. Use select structural Douglas fir.

4. Design a continuous timber-concrete composite deck to carry an H-20 loading when the bents are spaced 18 ft. apart and the roadway is 24 ft. wide. Use No. 1 dense structural southern pine.

5. Design a single span timber-concrete composite deck 22 ft. long to carry an H-15 loading when the roadway is 20 ft. wide. Use No. 1 dense structural southern pine.

CHAPTER VII

GLUED LAMINATED CONSTRUCTION

86. General. Glued laminated beams and arches have been used extensively in Europe and are rapidly gaining favor in this country. They are usually built up of nominal 1- or 2-inch material with each ply glued so that the grain of all the plies is in the same direction.

The method of gluing small boards together to form large cross sections has many advantages. Among these are (1) larger cross sections and lengths can be manufactured than those available as single pieces; (2) lumber that is not ordinarily classified as structural lumber can be used to form a cross section that will be just as strong as a solid piece; (3) the laminations can be arranged so that the highly stressed portions of a member will contain the pieces with the least number of defects and the greatest density; (4) a single cross section forming an arch rib has greater fire resistance than a truss constructed of smaller pieces designed to carry the same load; (5) in bowstring trusses the curved top chord can be made continuous through the panel points, thus simplifying fabrication. There is one chief disadvantage, which is found in the manufacturing process. In order to obtain the proper bond between the laminations, the amount of pressure and the temperature must be controlled during gluing operations. This means that for best results most members are fabricated in a shop and then shipped to the job site.

87. Manufacture. The builder of glued laminated members must have the proper equipment for mixing and spreading glue, for maintaining the required temperature, and for applying the necessary pressure. Each lamination is usually spread with glue on both sides by means of mechanical spreaders. However, this operation can be performed by hand. When curved members are being built it is necessary to bend the laminations around a suitable form. Pressure is applied by means of hand-operated clamps, screw jacks, hydraulic presses, or any other suitable device.

There are several methods used to join members end to end in order to get the required length. These are illustrated in Figure 87.

Butt joints have no strength in tension and are inadequate in resisting the bending of the laminations to the required curvature. Properly glued, the scarfed joint, in which the slope of the scarf is usually 1 in 12, is as strong as the wood itself. The hooked scarfed joint has the

advantage of providing quick assembly because the two pieces can be quickly joined without the ends overlapping. However, this joint is not as strong in tension as the plain scarfed joint.

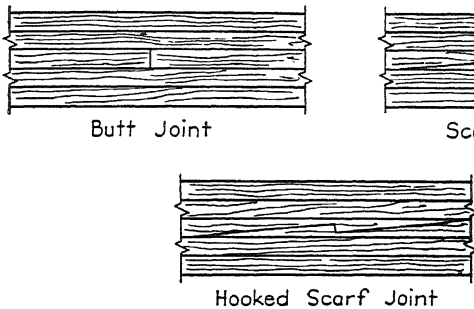


FIG. 87. Joints in laminations.

In arch construction, the arch rib is often tapered for architectural and economic reasons. There are several methods of accomplishing this taper, but the most practical is to have all the laminations running parallel to the center line of the member except the last one at each face, and it should follow the curvature of the face and be accurately fitted to the preceding laminations.

The cross section of the member can be built up to form a rectangle, I-beam, or box beam, but the most common type is the rectangle.

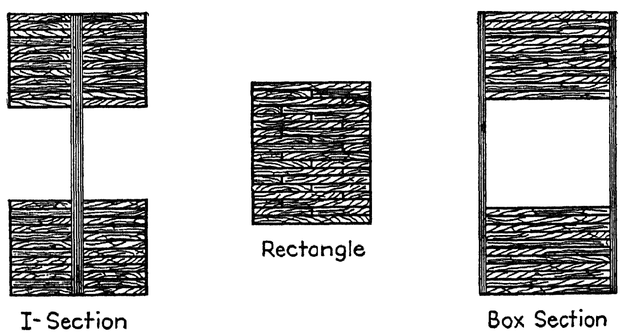


FIG. 88. Cross sections of glued laminated members.

The average maximum curvature for any lamination should not be more than that determined by multiplying the thickness of the laminations by 130. Of course, some species will permit bending to a smaller radius because the stress induced during bending is different for most species.

Because the fiber stresses in any member subjected to bending moments are greatest at the outermost fiber and gradually decrease to

zero at the neutral axis, it is possible to design a glued laminated member so that the innermost plies contain lumber having more defects than the face laminations. For purposes of assigning working stresses and of limiting the defects in the various laminations the Forest Products Laboratory suggests the division of the member into three volumes.

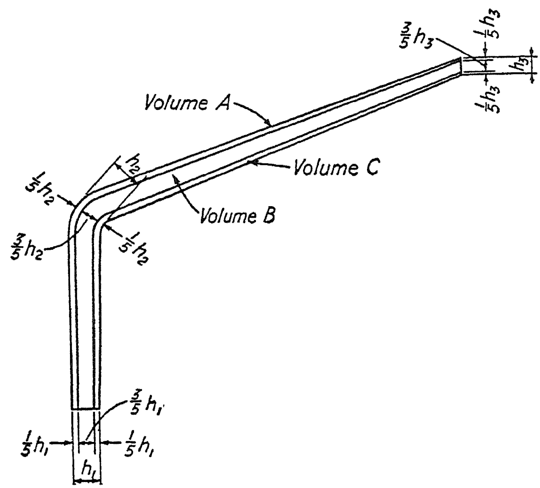


FIG. 89. Division of member into volumes.

Volumes A and C each occupy one-fifth of the cross section throughout the entire length of the member and volume B occupies the remaining portion.

88. Form Factor. Tests at the Forest Products Laboratory revealed that as the height of a beam increased the fiber stress at the proportional limit and the modulus of rupture decreased slightly. This makes it necessary to use a multiplier in the usual formula for internal resisting moment so that $M = \dfrac{FfI}{c}$, in which F is the form factor of the section. This form factor has already been considered in the unit working stresses for structural lumber; for rectangular sections it may be considered as unity. The formula for the resisting moment of a square wooden beam so oriented that a diagonal of the section is vertical requires a multiplier of 1.414 and the general formula is written $M = \dfrac{1.414fI}{c}$. For a circular section the form factor is 1.18.

For I and box forms the form factor is less than unity and at the proportional limit is given by the following equation:

$$F = 0.58 + 0.42\,[K(1 - t) + t]$$

The form factor for modulus of rupture is

$$F = 0.50 + 0.50\,[K(1 - t) + t]$$

t = ratio of total width of webs to the width of the section.
K = values depending on d, the ratio of the depth of the compression flange to the total depth.

The modulus of elasticity is unaffected by the shape of the cross section, and no form factor is required in computing deflections.

Table 29. Values of K for Different Values of d

d	K	d	K	d	K
0.10	0.085	0.45	0.660	0.80	0.985
.15	.155	.50	.740	.85	.995
.20	.230	.55	.810	.90	.998
.25	.315	.60	.875	.95	1.000
.30	.400	.65	.920	1.00	1.000
.35	.490	.70	.950		
.40	.575	.75	.970		

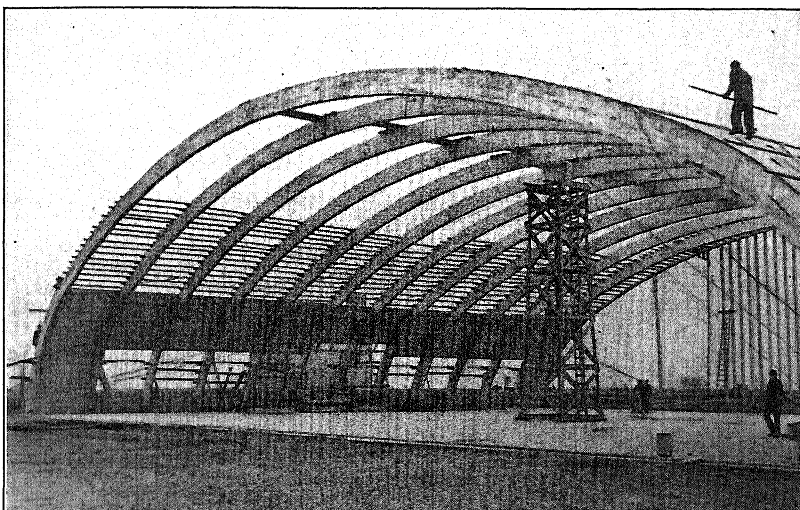


FIG. 90. 152-ft. glued laminated arches designed and fabricated by Unit Structures, Inc.

SPECIFICATIONS

The following specifications and design stresses have been prepared by the Forest Products Laboratory and appear in Bulletin 691 of the United States Department of Agriculture, under the title "The Glued Laminated Wooden Arch," by T. R. C. Wilson.

89. Moisture Content. Before being glued material must be seasoned to an average moisture content of not more than 15 per cent and not less than 10 per cent with a spread of not more than 5 per cent moisture content among pieces incorporated into a single built-up member.

90. Permissible Defects. Limitations of knots apply to both sides and limitations of slope of grain to both sides and both edges of the lamination. All laminations shall be free from shakes or splits that when viewed from the ends of the piece make an angle of less than 30° with the wide faces. Size of a knot is to be taken as the dimension of the knot between lines touching it and parallel to the edges of the lamination. Direction of grain is to be measured over a distance sufficiently great to determine the general slope, local deviations being disregarded. Material that is obviously so resinous as to be likely not to hold glue shall be rejected.

Laminations in Volumes *A* and *C*

Laminations in volumes *A* and *C* must not contain any part of the pith of the tree.

When laminations of Douglas fir, southern yellow pine, or redwood, are specified to be "close-grained" or "dense" they shall conform to the requirements for rate of growth or for rate of growth and percentage of summerwood as specified for close-grained or dense material of these species. Regardless of the grade of construction, the outer lamination on each side of the member shall be of the character specified for volumes *A* and *C* in grade I construction. Further, these laminations shall have a thickness not greater than one one-hundred-and-fiftieth of the minimum radius to which they are bent.

Grade I Construction

For grade I construction, full-width or part-width laminations in volumes *A* and *C* shall be free from knots whose size exceeds one-eighth the width of the piece—maximum size 1½ in. The sum of the widths of all knots in either face in any length equal to the width of the piece shall not exceed one-eighth the width. Laminations shall be free from diagonal or spiral grain whose slope is greater than 1 in 17. Wane whose greatest width does not exceed one-half the thickness of the piece is permissible.

Grade II Construction

For grade II construction, full-width or part-width laminations in volumes *A* and *C* shall be free from knots whose size exceeds one-fourth the width of the piece—maximum size 2½ in. The sum of the sizes

of all knots in either face within any length equal to the width of the piece shall not exceed one-fourth the width of the piece. Laminations shall be free from diagonal or spiral grain whose slope is greater than 1 in 15. Wane whose greatest width does not exceed the finished thickness of the piece is permissible.

Laminations in Volume *B*

Grades I or II Construction

Full-width or part-width laminations in volume *B* may have knotholes or sound knots whose combined widths do not exceed one-third the width of the lamination except that no defects shall be permitted that interfere with bending to the required curve without localized irregularities in the curvature, or that interfere with bringing laminations into close contact. Wane whose greatest width does not exceed the finished thickness of the lamination is permissible.

Laminations in Volumes *A*, *B*, and *C*

It may be specified that all laminations shall be free from knots, knotholes, or wane that will be visible when the member is in place in the structure.

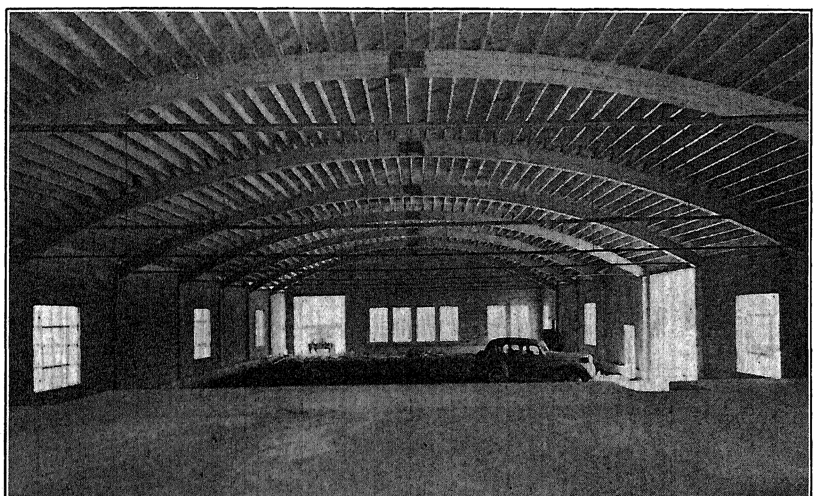


Fig. 91. Glued laminated arches spanning 58 ft. Fabricated by Rilco.

91. Preparation of Laminations. Laminations will be single piece in length or will be built up to full length of their runs by joining shorter

pieces end to end, prior to the final surfacing of either side, by means of glued plain scarf joints that cross the thickness of the piece in a distance not less than 12 times the thickness for joints in volumes A and C. In volumes A and C the sloping surfaces joined to form a scarf joint must be free from knots or pitch pockets.

92. Distribution of Joints in the Width and Length of Laminations. Outer laminations shall be of one piece in width. Other laminations may be of two or more pieces provided their widths and arrangement are such that longitudinal joints in adjacent laminations are separated by at least 1½ in. Laminations or part laminations composed of two or more pieces edge glued to each other prior to the final surfacing of either side may be considered as one piece.

Scarf joints in volumes A and C shall be so arranged that at any section perpendicular to the axis of the member the sum of the widths of the joints in any group of three successive laminations shall not exceed the width of the member. Furthermore, joints showing on either edge of a member shall not be closer together, center-to-center, in adjacent laminations in volumes A, B, or C than 24 times the thickness of a single lamination.

Joints in the length of members making up curved portions are to be avoided in so far as possible, and laminations in volumes A and B shall not be jointed at any point where the radius of curvature to which they will be bent is less than 125 times the thickness of the lamination.

93. Arrangement or Distribution of Taper. The tapering of members that vary in depth shall be accomplished by whichever of the following methods is specified.

(1) All laminations to parallel the center line of the member except the last one at each face, which shall follow the curvature of the face and shall be accurately fitted to the preceding laminations.

(2) A group of outer laminations totaling at least one-fifth the depth of the member at the point of maximum depth or one-half the depth at the point of minimum depth to run parallel to each face of the member, the remaining laminations being so arranged that fitting their ends will not require cutting at a slope steeper than 1 in 12. Such fitting must be accurately done so that a good glue joint results.

(3) The total taper to be approximately uniformly divided among all laminations, i.e., each lamination to be tapered in the same proportion as the member itself. For example, if there are 20 laminations, each will have at each point in its length a thickness approximately one-twentieth the depth of the member at the corresponding point.

94. Machining. All surfaces to be joined by gluing shall be carefully machined so that they are true and free from humps or depressions, and at the time of gluing they shall be free of dust, dirt, or grease.

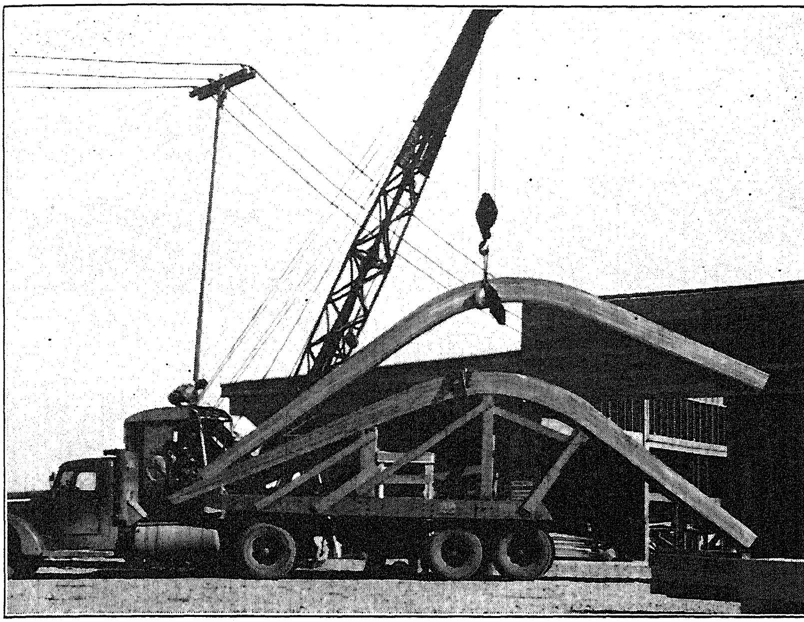


FIG. 92. Glued laminated arch segments loading at Timber Structures, Inc., for shipment to Camp White, Medford, Oregon.

95. Glue. Only a water-resistant casein glue, or other glue known to be equal in strength and moisture resistance to best quality casein glue, is to be used. Glue must be thoroughly mixed and must be free from lumps and from excessive air bubbles.

Not less than 8 lb. of wet casein glue per 100 sq. ft. of joint area, or equivalent amounts of other types used, shall be applied. Glue shall be applied to a uniform thickness and preferably to both of the faces meeting at a joint. Glue-coated pieces shall be laid together as soon as the glue is spread.

96. Pressure. While the glue is setting all glued joints are to be subjected to a pressure of not less than 100 nor more than 200 lb. per sq. in. by means of clamps, screw jacks, presses, or other similar appliances.

Clamping must be completed within 20 min. after the first glue is spread if glue is applied to both faces meeting at a joint, or within 15 min. if glue is applied to only one of these faces. When a member is built up in installments, each group or installment of laminations shall remain under pressure for at least 3 hr. before being released to add the next installment. Pressure shall be maintained at least 12 hr. after the addition of the last installment.

Special care must be taken to assure good gluing between successive installments of laminations.

97. Temperature. The room in which gluing is done shall be maintained at a temperature of 50° F. or higher. Material is to be brought approximately to the temperature of the glue room before gluing is begun.

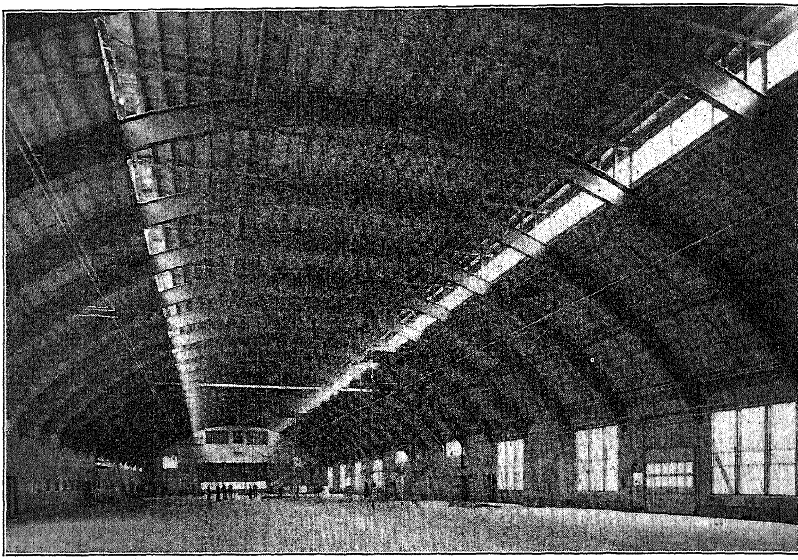


FIG. 93. Laminated arches used in navy drill halls. Fabricated by Unit Structures, Inc., Peshtigo, Wisconsin.

DESIGN STRESSES

The following is suggested as the procedure for determining from Table 30 the stresses for use in the design of laminated members built in accordance with the specifications presented in the preceding section.

98. Combined Bending and Compression. The sum of the bending stress and the compressive stress parallel to the axis of the member shall not exceed the value given in Table 30, Column 2 (increased in accordance with Footnote 1 when this footnote is applicable to the material specified for use in volumes A and C), multiplied by grade of construction, curvature, and depth factors as follows:

Grade of construction factors:

For grade I construction	1.000
For grade II construction	0.875

Curvature factor $\quad 1.00 - 2.00\left(\frac{t}{R}\right)^2$

Table 30. Basic Stress Values for Laminated Wooden Construction in Pounds per Square Inch [1]

Species (1)	Combined Bending and Compressive Stress in Extreme Fiber (2)	Compression Perpendicular to Grain (3)	Compression Parallel to Grain (4)	Maximum Longitudinal Shear (5)	Modulus of Elasticity (6)
Softwoods:					
Cedar, Alaska	1,466	250	1,066	120	1,200,000
Cedar, northern and southern white	1,000	175	733	93	800,000
Cedar, Port Orford	1,466	250	1,200	120	1,500,000
Cedar, western red	1,200	200	933	106	1,000,000
Cypress, southern	1,733	300	1,466	133	1,200,000
Douglas fir, coast region	2,000	325	1,466	120	1,600,000
Douglas fir, Rocky Mountain region	1,466	275	1,066	113	1,200,000
Fir, commercial white	1,466	300	933	93	1,100,000
Fir, balsam	1,200	150	933	93	1,000,000
Hemlock, eastern	1,466	300	933	93	1,100,000
Hemlock, western [2]	1,733	300	1,200	100	1,400,000
Pine, western white,[3] northern white, sugar, and ponderosa	1,200	250	1,000	113	1,000,000
Pine, red	1,466	300	1,066	113	1,200,000
Pine, southern yellow [4]	2,000	325	1,466	146	1,600,000
Pine, southern yellow, dense	2,333	380	1,711	171	1,600,000
Redwood	1,600	250	1,333	93	1,200,000
Spruce, Engelmann	1,000	175	800	93	800,000
Spruce, red, white, and Sitka	1,466	250	1,066	113	1,200,000
Tamarack	1,600	300	1,333	126	1,300,000
Hardwoods:					
Ash, commercial white	1,866	500	1,466	167	1,500,000
Ash, black	1,333	300	866	120	1,100,000
Beech	2,000	500	1,600	167	1,600,000
Birch, sweet and yellow	2,000	500	1,600	167	1,600,000
Chestnut	1,266	300	1,066	120	1,000,000
Elm, rock	2,000	500	1,600	167	1,300,000
Elm, American and slippery [5]	1,466	250	1,066	133	1,200,000
Gum, black and red	1,466	300	1,066	133	1,200,000
Hickory, true and pecan	2,533	600	2,000	187	1,800,000
Maple, sugar and black [6]	2,000	500	1,600	167	1,600,000
Oak, commercial red and white	1,866	500	1,333	167	1,500,000
Tupelo	1,466	300	1,066	133	1,200,000

[1] For members with laminations in volumes *A* and *C* of close-grained material of Douglas fir from the Pacific Coast region, southern yellow pine, or redwood, values in columns (2), (3), and (4) may be increased 1/15. Close-grained material is defined as follows: Douglas fir from the Pacific Coast region and southern yellow pine shall average on one end or the other of the piece not less than 6 nor more than 20 annual growth rings per radial inch. Pieces averaging from 5 to 6 annual rings per inch to be accepted as the equivalent of close grained if having one-third or more summerwood. Redwood shall average on one end or the other not less than 10 nor more than 25 annual growth rings per radial inch. For members with laminations in volumes *A* and *C* of dense Douglas fir or dense southern yellow pine, values in columns (2), (3), (4), and (5) may be increased 1/6. Dense material of these species shall average on one end or the other not less than 6 annual growth rings per radial inch and in addition not less than one-third summerwood.

[2] Also sold as west coast hemlock. [3] Also sold as Idaho white pine. [4] Also sold as longleaf or shortleaf southern pine. [5] Also sold as white elm or soft elm. [6] Also sold as hard maple.

where t/R is the maximum value of thickness of lamination divided by the radius to which the lamination is bent at that point in the length of the member at which the stress occurs. (No curvature factor is to be applies to stress in a straight portion of a member regardless of the curvature elsewhere.)

$$\text{Depth factor, } 1.07 - 0.07 \sqrt{\frac{h}{2}}$$

where h is the depth of the member in inches at the point under consideration.

If I- or box-sections are used, further multiplication by the appropriate form factor should be made.

99. Compression Parallel to Grain. The allowable stresses in compression parallel to the grain are:

For laminations of the type specified for volumes A and C in grade I construction: The value given in Table 30, Column 4, increased for close-grained or dense material according to Footnote 1.

For laminations of the type specified for volumes A and C in grade II construction: 80 per cent of the value given in Table 30, Column 4, increased for close-grained or dense material according to Footnote 1.

For laminations of the type specified for volume B: 75 per cent of the value given in Table 30, Column 4, without increase for close-grained or dense material.

The total thrust at any cross section shall not exceed the sum of the products of the cross-sectional areas of laminations of each type multiplied by the allowable stress in compression parallel to grain for laminations of that type.

100. Compression Perpendicular to Grain (at Bearings or Radial Compression). For allowable stresses in compression perpendicular to grain, the value taken from Table 30, Column 3, increased for close-grained or dense material according to Footnote 1, should be used.

101. Tension Perpendicular to Grain (Radial Tension). For allowable stresses in tension perpendicular to grain, the values taken from Table 30, Column 3, should be used when multiplied by one-seventh for softwood species or one-sixth for hardwood species and further multiplied by the following factors according to the type of material at the point under consideration:

0.70 for defects restricted as specified for volumes A and C in grade II construction; 0.60 for defects restricted as specified for volume B.

102. Longitudinal Shear. For allowable stresses in longitudinal shear, three-fourths of the values in Table 30, Column 5, should be used.

103. Design. Two-hinged and three-hinged arches are the types to which glued laminated construction is best adapted. Because of the ease in fabrication and erection, the majority are designed as three-hinged. The following example illustrates the method of obtaining the stresses and the dimensions of the cross section at several points in a typical three-hinged arch uniformly loaded. The writer has found by experiment that the uniform loads may be considered concentrated every 6 ft. or less without any appreciable error. This greatly reduces the work involved in obtaining the maximum stresses. However, in the following example the maximum moments, thrusts, and shears have been determined graphically by considering the dead load uniformly distributed across the span, and the live load uniformly distributed on the full span and on the half span.

Table 31. Maximum Design Stresses

Section	Moment (ft.-lb.)	Thrust (lb.)	Shear (lb.)
a	47,500	13,780	7,950
b	85,000	15,850	2,140
c	49,600	11,680	7,880
d	21,360	10,460	4,470
e	15,140	9,130	1,650

Assuming a width of 7½ in. and using close-grained 1-in. southern pine boards of a grade equal to grade I construction, consider the design of the section at point *b*.

Assume $d = 28.5$ in. $\qquad b = 7.5$ in.

Allowable $f = 2130 \times$ construction factor $\times$ curvature factor $\times$ depth factor.

Construction factor $= 1.00$.

$$\text{Curvature factor} = 1.00 - 2000\left(\frac{25/32}{72 - 14.25}\right)^2 = 0.634.$$

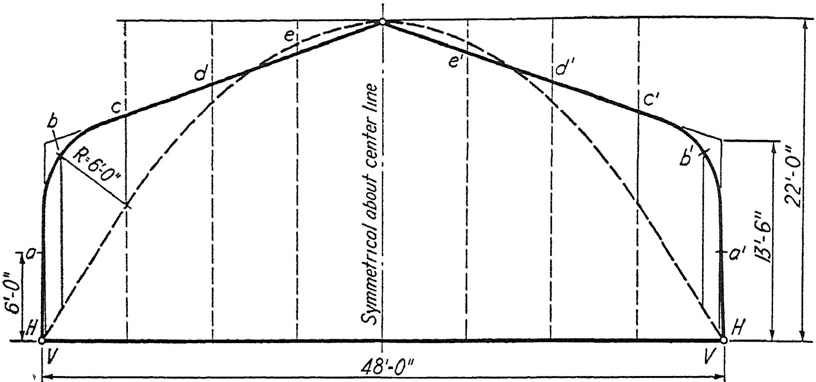
Depth factor $= 1.07 - 0.07\sqrt{14.25} = 0.806$.

Allowable $f = 2130 \times 0.634 \times 0.806 = 1086$ lb. per sq. in.

$$\text{Actual } f = \frac{T}{A} + \frac{Mc}{I} = \frac{15{,}850}{7.5 \times 28.5} + \frac{85{,}000 \times 12 \times 6}{7.5 \times 28.5 \times 28.5} = 1077 \text{ lb. per sq. in.}$$

Allowable shear $= 146 \times 3/4 = 110$ lb. per sq. in.

$$\text{Actual shear} = \frac{3}{2} \times \frac{2140}{7.5 \times 28.5} = 15 \text{ lb. per sq. in.}$$



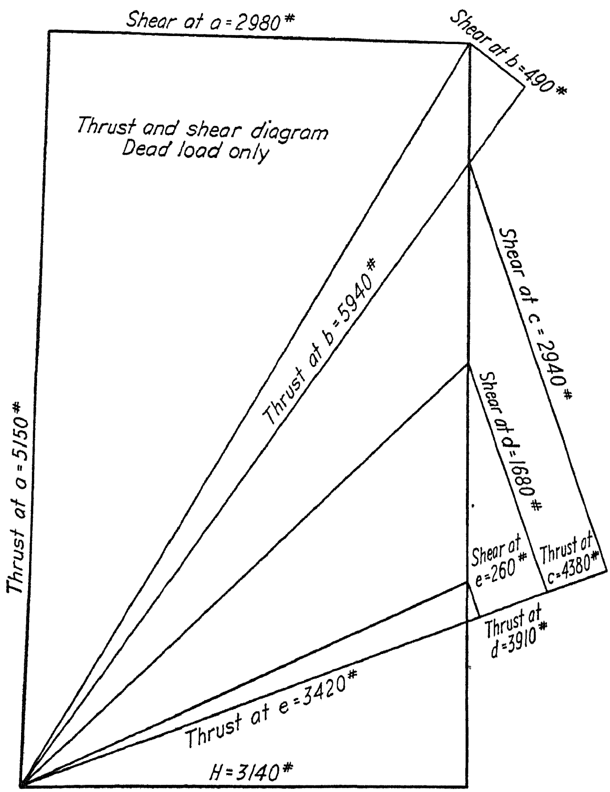


FIG. 94. Dead load stress diagram.

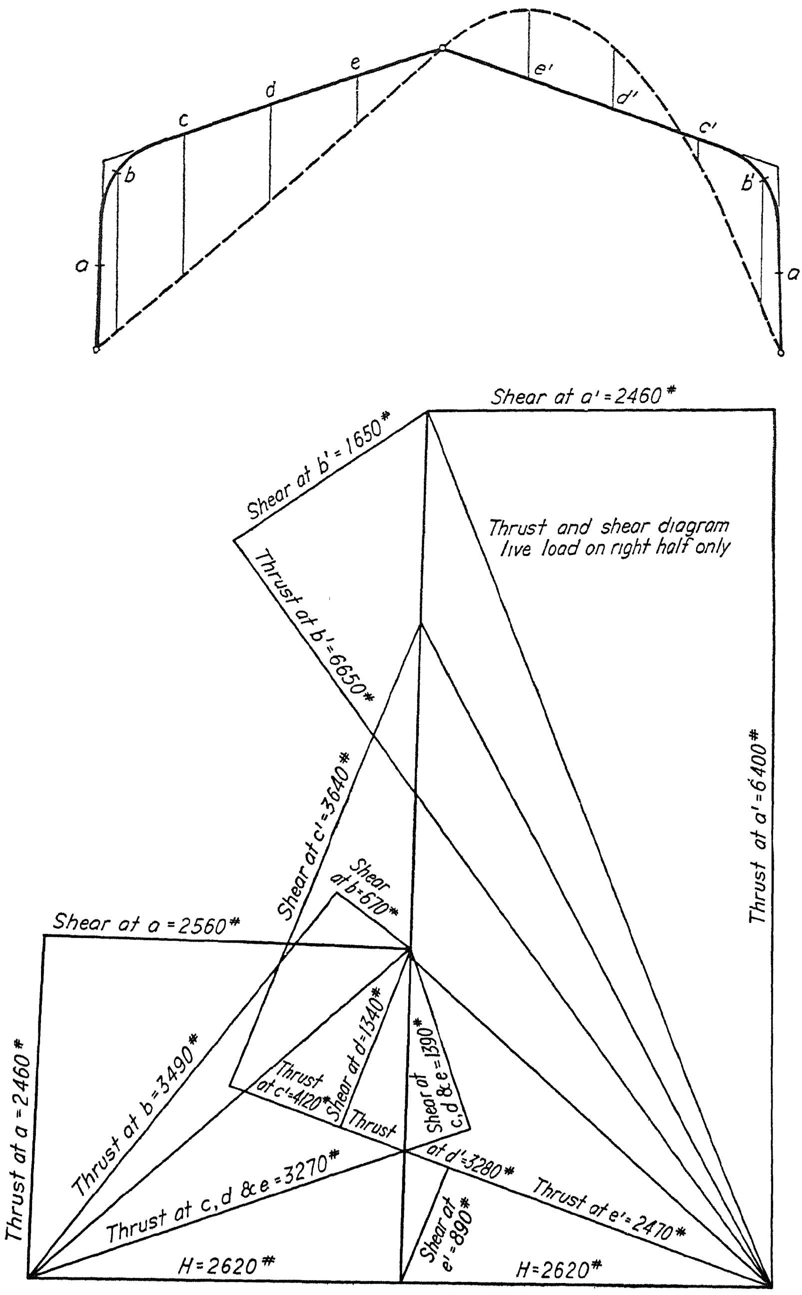


FIG. 95. Stress diagram for live load on one-half the span.

It is impossible to make the final cross sections of the arch rib the exact size as determined from the maximum stresses. For architectural and practical reasons the shape of the arch rib must be modified so that a uniform taper results. This means that some sections will have very low stresses.

Table 32. Required and Final Sizes

Section	Width (in.)	Depth (in.) Required	Depth (in.) Used
a	7.5	15.50	15.50
b	7.5	28.50	28.50
c	7.5	16.50	16.50
d	7.5	10.75	15.00
e	7.5	9.00	9.00

104. Radial Stress in a Curved Member. The exact solution [1] of the stresses in a curved beam subjected to bending moments shows that the tangential forces produce resultants in a radial direction that tend to separate the fibers when the moment is in a direction causing tension on the concave side and compression on the convex side. When the moment is in the opposite direction the radial stress is in compression. The radial stress will always be less than the fiber stress and will be a maximum near the neutral axis.

The design stresses for the fiber stresses due to bending moments in a curved laminated member have been derived on the assumption that linear distribution of bending stress exists. Of course, this assumption is incorrect, a fact recognized by Mr. Wilson; [2] but because the design stresses are based on this assumption, it is believed that very little error will result by using a linear distribution of bending stress in determining the maximum fiber stress due to external loads. This value, which we shall call f_x, is less than the true stress.

In determining the radial stress it is necessary to know the location in the member where the maximum radial stress occurs and then calculate the radial stress by the formula obtained from the exact solution of the stresses existing in a curved member. This method is rather laborious, and for convenience the following chart has been prepared.

The factor K is a combination of two constants multiplied together. The fact that the fiber stress obtained when a linear distribution of stress is assumed is less than the actual maximum fiber stress makes it necessary to multiply by a constant to determine the true stress. This con-

[1] *Theory of Elasticity*, by S. Timoshenko, p. 58, 1934.

[2] "The Glued Laminated Wooden Arch," by T. R. C. Wilson, U. S. Dept. Agr., Bull. 691, 1939.

stant depends upon the ratio of the radius of curvature to the depth of the member. The radial stress is some percentage of the true stress and also depends upon the ratio of the radius of curvature and the depth of the member. K, then, combines these two constants, and becomes a value to be used as a multiplier of f_x, the fiber stress based on a linear distribution of bending stress. The radial stress f_r is equal to Kf_x.

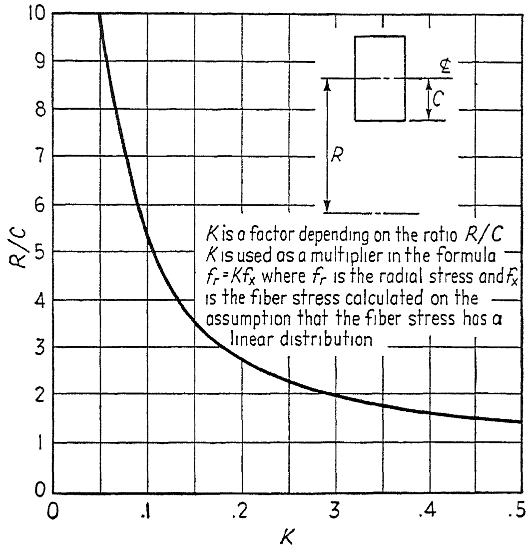


CHART 15. Values of K for determining the radial stress in a curved member subjected to bending moments.

The radial stress at point b in the previous example is in compression and acts perpendicular to the grain of the member. It may be calculated as follows:

$$R = 72 \text{ in.} \qquad c = 14.25 \text{ in.}$$

$$\frac{R}{c} = \frac{72}{14.25} = 5.05$$

For this value, K from Chart 11 is 0.106

$$f_r = Kf_x = 0.106 \times 1003 = 106.3 \text{ lb. per sq. in.}$$

The allowable compression perpendicular to the grain = 346 lb. per sq. in.

105. Typical Details. The following figures illustrate the methods used in joining laminated members at the crown and at the walls or abutments.

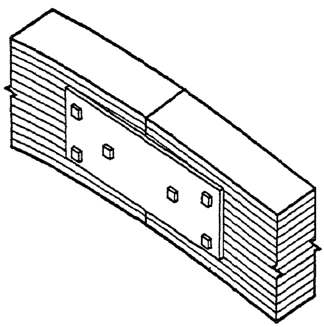


FIG. 96. Typical crown joint.

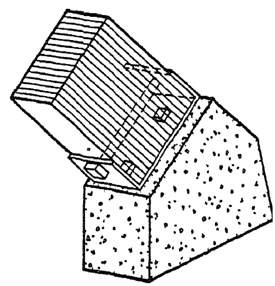


FIG. 97. Method of attaching member to abutment.

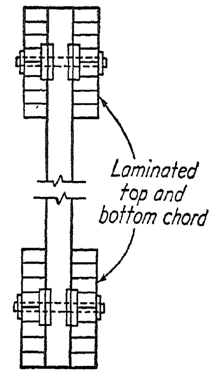


FIG. 98. Cross section of a bowstring truss.

PROBLEMS

1. Design a glued laminated beam to span 30 ft. and to carry a uniformly distributed load of 1000 lb. per ft. Use close-grained western hemlock and assume the thickness of each lamination as 1 in. (nominal).

2. A glued laminated beam simply supported between two points 40 ft. apart carries a concentrated load of 1000 lb. at the center and a uniform load of 800 lb. per ft. The beam is made of 1 by 12 in. close-grained Douglas fir boards. How many boards are required?

3. A laminated member is built up to a radius of 8 ft. The laminations are 1 by 8 in. dense southern pine and the member is subjected to a bending moment of 95,000 ft.-lb., a thrust of 17,000 lb., and a shear of 3000 lb. How many laminations are required?

4. Design a three-hinged glued laminated arch to span 70 ft. with a rise of 9 ft. 6 in. and a radius of 73 ft. to support a total load of 40 lb. per sq. ft. of horizontal surface. The arches are spaced center-to-center every 16 ft. Use 1-in. close-grained Douglas fir.

5. Design a three-hinged glued laminated arch to span 60 ft. with a total rise of 24 ft. The side legs are 14 ft. high. The arch is to carry a wind load of 20 lb. per sq. ft. of vertical surface, snow load of 20 lb. per sq. ft., and dead load of 15 lb. per sq. ft. of horizontal surface. The arches are spaced 12 ft. apart. Use 1-in. close-grained southern pine.

6. Using a top chord of two members built up of 2-in. close-grained Douglas fir and split ring connectors design a bowstring truss to carry a total load of 40 lb. per sq. ft. of horizontal surface. The trusses are spaced 16 ft. apart and span 80 ft.

CHAPTER VIII

PLYWOOD

106. General. Plywood in its simplest form consists of three layers of thin veneer glued together with the grain of the middle ply perpendicular to the grain of the face plies. The advantages in laying up veneers with the grain of adjacent plies perpendicular to each other are apparent when we consider that wood has its greatest strength along the grain. The plywood panel tends to equalize the strength in both directions and has a very high resistance to splitting by nails, screws, or other fastenings.

Because wood shrinks more across the grain and very little along the grain plywood panels are made up of an uneven number of veneers. If this were not done the panels would warp considerably. The two outside plies are called faces, or face and back, and the center ply is called the core. Any intervening plies are cross bands or inner bands.

Plywood can be made from a variety of species that can be combined to make the best product for the use intended. Moreover, the direction of the grain in the various plies can be made at any angle to fit the needs of design, and almost any curvature can be obtained. At present plywood is used in the manufacture of furniture, boxes and crates, doors, boats, and many industrial products. In the structural field it is used primarily as floors, partitions, sheathing, as forms for concrete, gusset plates in large timber structures, and as webs for plywood girders and arches. In the aircraft industry the use of plywood seems to be unlimited as it is being used to advantage for wing and fuselage coverings, leading edges, gusset plates, ribs, webs in box beams, nose and tail pieces, floors, walls, and numerous other parts.

MANUFACTURE

107. Veneers. In manufacturing plywood it is first necessary to cut the veneers to the desired thickness. Most veneers are rotary cut by placing a steamed log on a rotating lathe. As the lathe revolves knives peel off a continuous layer of wood between $\frac{1}{40}$ in. and $\frac{1}{4}$ in. in thickness. These are customary thicknesses, but thinner or thicker veneers can be cut, depending upon the species of wood. Veneers are also sliced off the log when a figured pattern or edge-grained material is desired.

After the veneers are cut they are stacked and dried to a moisture content of approximately 10 per cent. Then they are flattened, and the edges trimmed or clipped so that the smaller pieces can be taped together before assembling them into plywood.

The weight of veneers depends upon the species, thickness, and moisture content. Table 33 may be used as a guide in calculating the weight of a plywood panel.

108. Glues. The oldest form of glue is that made from the hides and bones of animals. This type is not used to any extent in the manufacture of plywood because the better grades are expensive and the glue is not water resistant.

Vegetable glues are made from starch either with or without certain chemicals added. The mixtures are relatively cheap and can be used cold; however, they are not used very often in making plywood because they are not water resistant and often cause discoloring of certain species.

Casein glues, which are made from the curd of milk mixed with lime or sodium salt, have a high degree of water resistance. They are mixed and applied when cold and are particularly useful in the construction of glued laminated members. Practically all members of this type have been made with some form of casein glue. These glues are also used for certain forms of plywood.

The blood albumin glues made by combining the blood of animals with chemicals, such as lime, caustic soda, and sodium silicate, were the first glues requiring heat to make them set. These glues are fairly waterproof but have the disadvantages of staining certain veneers, emitting an objectionable odor, and of producing joints of a low dry strength.

The recent development of the phenolic resin glues has revolutionized the plywood industry. Two types of these glues are manufactured, the phenol formaldehyde and the urea formaldehyde. Both are furnished either as a liquid or as dry powder. The phenol resins are more durable than the urea resins. Both are, however, extremely durable under severe conditions of exposure and also possess a high resistance to mold or fungus growth. They are boilproof and can be soaked in water indefinitely and in almost every case will outlast unprotected wood.

The phenol resins have the added advantage of being made as a continuous film. This greatly reduces the work involved in laying up veneers for the manufacture of plywood. Moreover, extremely thin veneers are difficult to spread with a liquid adhesive, but the thin film of phenol resin can be readily placed between them.

109. Heat and Pressure. The most common form of applying heat and pressure simultaneously in the manufacture of plywood is by means of a hot plate press which consists of a number of platens through which

Table 33. Weights of Veneers

[Ounces per Square Foot, Based on Oven-Dry]

Species	Specific Gravity Oven-Dry Weight and Volume at 12% Moisture Content	Veneer Thickness, in Inches																	
		1/100	1/80	1/64	1/60	1/55	1/48	1/40	1/32	1/28	1/24	1/20	1/16	1/12	1/10	1/8	1/6	3/16	1/4
Basswood	0 37	0 31	0.38	0.48	0 51	0 56	0 64	0 77	0 96	1.10	1 28	1 54	1.92	2 57	3 08	3 85	5 13	5 77	7 70
Beech	.62	.52	.64	.81	.86	.94	1.07	1 29	1 61	1 84	2.15	2 58	3 22	4 30	5 16	6 45	8 60	9 67	12 90
Birch, sweet	.65	.54	.68	.84	.90	.98	1.13	1.35	1.69	1.93	2.25	2 70	3.38	4 51	5 41	6 76	9.02	10.14	13.52
Birch, yellow	.62	.52	.64	.81	.86	94	1.07	1 29	1 61	1 84	2.15	2 58	3.22	4.30	5 16	6 45	8 60	9 67	12 90
Cedar, Port Orford	.42	.35	.44	.55	.58	.64	0.73	0.87	1 09	1.25	1.46	1.75	2.18	2.91	3.49	4.37	5 82	6 55	8 74
Douglas fir (coast region)	.48	.40	.50	.62	.67	.73	.83	1 00	1.25	1 43	1 66	2.00	2 50	3.33	3.99	4.99	6 66	7 49	9 98
Gum, red	.49	.41	.51	.64	.68	.74	.85	1.02	1.27	1 46	1.70	2 04	2 55	3.40	4 08	5.10	6 80	7 64	10 19
Gum, tupelo	.50	.42	.52	.65	.69	.76	.87	1 04	1 30	1.49	1.73	2 08	2 60	3 47	4 16	5 20	6 94	7 80	10 40
Magnolia, southern	.50	.42	.52	.65	.69	.76	.87	1 04	1.30	1.49	1.73	2 08	2 60	3 47	4 16	5 20	6 94	7 80	10.40
Mahogany, African	.45	.37	.47	.58	.62	.68	.78	0 94	1 17	1 34	1.56	1.87	2.34	3.12	3 74	4 68	6 24	7 02	9 36
Mahogany, Central American	.46	.38	.48	.60	.64	.70	.80	96	1 20	1.37	1 60	1 91	2.39	3.19	3.83	4.78	6.38	7.18	9 57
Maple, soft	.47	.39	.49	.61	.65	.71	.81	.98	1.22	1.40	1.63	1.96	2.44	3.26	3.91	4 89	6 52	7.33	9 78
Maple, hard	.63	52	.66	.82	.87	.95	1.09	1.31	1 64	1.87	2 18	2.62	3.28	4.37	5.24	6 55	8 74	9.83	13.10
Spruce, red, white, Sitka	.39	.32	.41	.51	.54	.59	0.68	0 81	1 01	1 16	1 35	1 62	2 03	2 70	3.24	4.06	5 41	6 08	8 11
Walnut, black	.55	.46	.57	.72	.76	.83	.95	1.14	1 43	1.63	1 91	2 29	2.86	3 81	4.58	5 72	7 63	8 58	11 44
Yellow poplar	.40	33	.42	52	.55	61	.69	0.83	1.04	1.19	1 39	1.66	2 08	2.77	3.33	4 16	5 55	6 24	8 32

NOTES: The weights for any given moisture content can be found by multiplying the weights given above by 1 plus the particular moisture content.

In calculating the weight of plywood for any combination of veneers, the weight of each glue line must be added. The approximate weight of phenolic resin glue per square foot of single glue line is 0 2 ounce.

Example: To find the weight of a 3-ply panel consisting of 2 face plys of 1/32-in. basswood and a core of 1/20-in. poplar glued with phenolic resin film at a moisture content of 12 per cent proceed as follows:

$$\text{Weight per square foot} = [2 \times 0.96 + 1 \times 1.66]\ 1.12 + 2 \times 0.2 = 4\ 41 \text{ ounces}$$

Table 34. Comparison of the Properties of Various Glues[1]

[Glues for use in aircraft, Bulletin No. 1337, Forest Products Laboratory]

Property or Characteristic	Casein Glue	Blood Albumin Glue	Synthetic Resin Glue
Strength (dry)[2]	Very high to high	High to low	Very high to high
Strength (wet after soaking in water 48 hr.).[3]	About 25 to 50 per cent of dry strength—varies with glue.	About 50 to nearly 100 per cent of dry strength.	Very high; nearly 100 per cent of dry strength.
Durability in 100 per cent relative humidity or prolonged soaking in water.	Deteriorates eventually—rate varies with glue.	Deteriorates slowly but usually completely in time.	Very high if resin is unadulterated.
Rate of setting	Rapid	Very fast with heat	Very fast with heat
Working life	Few hours to a day	Few to many hours	Few to several hours for liquid forms; several weeks for films.
Consistency of mixed glue	Medium to thick; little change with temperature.	Thin to thick; little change with temperature.	Medium for liquid forms
Temperature requirements	Unimportant	Heat required to set most glues	Heat required for most glues
Mixing and application	Mixed cold with water; applied cold by hand or mechanical spreaders.	Usually mixed cold with water; applied cold by hand or mechanical spreaders.	Often applied as received or after addition of "catalyst"; liquid forms best applied by rubber-covered rolls.
Tendency to foam	Slight if not mixed too rapidly.	Slight to pronounced	Slight
Tendency to stain wood	Pronounced with certain woods	None, except dark glue may show through thin veneers.	None, although glue may penetrate through thin or porous veneers.
Dulling effect on tools	Moderate to pronounced	Slight	Moderate
Spreading capacity,[4]			
Extremes reported[5]	35 to 80 per cent	30 to 100 per cent	30 to 100 per cent
Common range[5]	40 to 60 per cent	....	35 to 50 per cent

[1] Grades and quality only of glues that pass aircraft specifications.
[2] Based chiefly on joint strength tests.
[3] Based on plywood strength tests.
[4] Expressed in square feet of single glue line per pound of dry glue for veneer work.
[5] Based on reports from manufacturers of various commercial products.

steam is circulated. These presses usually have openings 50 by 100 in. which produce finished flat panels 4 by 8 ft. The pressures used range from 150 to 300 lb. per sq. in. and the temperatures range from 230° to 320° F. depending upon the species.

Pressures can also be applied by using flexible bags in conjunction with molding forms or dies. These bags can be used on the outside or inside of a mold to form such curved parts as leading edges for airplane wings or boat hulls. The medium of pressure is usually in the form of hot water within the bag with additional heat circulating in the wooden or steel mold. This process has particular advantages for the manufacture of curved plywood because the veneers can be bent before they are glued, thus eliminating the stresses that are ordinarily set up in plywood that is bent after manufacture.

A recent development in the manufacture of plywood is known as a high-frequency method. In this process the heat required to produce the bond between the veneers is created by an electric current oscillating at a very high frequency. Since the capacity of a condenser is measured by the amount of electricity that it is capable of storing, and since this amount varies with the kind of dielectric between the plates, it is readily conceived that wood will absorb more energy in the form of heat when it is wet than when it is dry because dry wood has a dielectric constant of 3.6 and water has a dielectric constant of 81. This also means that the greatest heating effect will be in the wettest portion of the wood, and in making plywood the glue line is the wettest portion. By the high-frequency method the wet glue can be made hotter than the wood and danger to the wood is eliminated because the over-drying of veneers during the short time required to cure the resin bond is improbable. In the manufacturing operation the electrodes are placed on opposite sides of the plywood assembly and the pressure is provided mechanically. Up to the present this manufacturing process has not been thoroughly investigated, but indications point to an increase in the speed of manufacturing thick panels and in adaptions for use in making molded parts.

110. High Density Plywood. The use of hot-pressed thermosetting resin adhesives for the manufacture of waterproof plywood requires pressures that seldom exceed 200 lb. per sq. in., and the total compression of the wood or reduction in thickness is rarely more than 10 per cent. Recently, however, the possibility of increasing the pressures used in the manufacture of resin-bonded plywood has opened up an entirely new field. Tests have been made on plywood subjected to pressures of 500 to 1500 lb. per sq. in., and today several manufacturers are making this high density or compressed plywood.

By an increase of the pressure applied in the hot plate process a reduction in thickness results which means that there is more wood fiber per square inch of cross section, and the compressed product will be stronger per square inch than ordinary plywood. The strength-weight ratio of compressed plywood makes it an ideal material for use in airplane construction. Its properties can be used to advantage in the design of reinforcing plates for the attachment of spars to the fuselage, gusset plates for ribs, hinges, floors, instrument panels, and seats. Many other industrial uses are possible, such as ribs and stems for boats, gun stocks, implement handles, gears, and dies.

It is also possible to manufacture a variable density plywood, i.e., one that has extra laminations at one end. In this type one end will be stronger than the other, but the entire panel will have the same thickness. The finished product has distinct advantages in propeller construction because the high density section can be attached at the hub where strength is needed, and the low density section toward the tip of the propeller reduces centrifugal forces.

Compressed plywood is manufactured in the same manner as ordinary plywood, except higher pressures are used. It is also possible to make an impregnated plywood. In this type the veneers are soaked in a phenolic resin solution until the desired impregnation is obtained. Of course, the time required for impregnation can be reduced by mechanical means. After impregnation the surplus resin is evaporated and the veneers are assembled into plywood under heat and pressure. Lower pressures can be used with impregnated veneers to obtain the same amount of compression as that developed in resin film plywood. However, the manufacture of compressed plywood with liquid resin impregnates is more complex and requires more time and care.

DOUGLAS FIR PLYWOOD

111. Sizes. Douglas fir is the species used for manufacturing most plywood that is intended for sheathing, flooring, forms for concrete, gusset plates in timber trusses and arches, and for webs in plywood girders. It is available in standard sizes up to 48 in. wide and 96 in. long and in various thicknesses of $\frac{3}{16}$ to $1\frac{3}{16}$ in. and scarf-jointed smaller panels of practically any size. There are also a few manufacturers capable of producing panels 8 ft. wide and 16 ft. long.

112. Grades. Douglas fir plywood is divided into two general classes depending upon the use for which the panel is intended. One type is made with a phenolic resin adhesive and is meant for exterior use because it is waterproof. The other is made with an ordinary moisture-resistant

Table 35. Veneer Thicknesses for Douglas Fir Plywood Panels

[R, rough; S, sanded]

Plywood Thickness (net)	No. of Plies	Veneer Thickness (Nominal, in Inches)		
		Faces	Centers	Cross Band
$\frac{1}{8}''$—R	3	$\frac{1}{24}$	$\frac{1}{24}$	
$\frac{1}{8}''$—S	3	$\frac{1}{16}$	$\frac{1}{16}$	
$\frac{3}{16}''$—R	3	$\frac{1}{16}$	$\frac{1}{16}$	
$\frac{3}{16}''$—S	3	$\frac{1}{12}$	$\frac{1}{12}$	
$\frac{1}{4}''$—R	3	$\frac{1}{12}$	$\frac{1}{12}$	
$\frac{1}{4}''$—S	3	$\frac{1}{9}$	$\frac{1}{9}$	
$\frac{5}{16}''$—R	3	$\frac{1}{10}+$	$\frac{1}{10}+$	
$\frac{5}{16}''$—S	3	$\frac{1}{8}$	$\frac{1}{8}$	
$\frac{3}{8}''$—R	3	$\frac{1}{8}$	$\frac{1}{8}$	
$\frac{3}{8}''$—S	3	$\frac{1}{8}$	$\frac{3}{16}$	
$\frac{7}{16}''$—R	3	$\frac{1}{8}$	$\frac{3}{16}$	
$\frac{7}{16}''$—S	5	$\frac{1}{10}$	$\frac{1}{10}$	2 @ $\frac{1}{10}$
$\frac{1}{2}''$—R	5	$\frac{1}{10}$	$\frac{1}{10}$	2 @ $\frac{1}{10}$
$\frac{1}{2}''$—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{10}$
$\frac{9}{16}''$—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{10}$
$\frac{9}{16}''$—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{8}$
$\frac{5}{8}''$—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{1}{8}$
$\frac{5}{8}''$—S	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{1}{8}$
$\frac{11}{16}''$—R	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{1}{8}$
$\frac{11}{16}''$—S	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{3}{16}$
$\frac{3}{4}''$—R	5	$\frac{1}{8}$	$\frac{1}{8}$	2 @ $\frac{3}{16}$
$\frac{3}{4}''$—S	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{3}{16}$
$\frac{13}{16}''$—R	5	$\frac{1}{8}$	$\frac{3}{16}$	2 @ $\frac{3}{16}$
$\frac{13}{16}''$—S	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{1}{8}$
$\frac{7}{8}''$—R	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{1}{8}$
$\frac{7}{8}''$—S	7	$\frac{1}{8}$	2 @ $\frac{5}{32}$	3 @ $\frac{1}{8}$
$\frac{15}{16}''$—R	7	$\frac{1}{8}$	2 @ $\frac{5}{32}$	3 @ $\frac{1}{8}$
$\frac{15}{16}''$—S	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{1}{8}$
$1''$—R	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{1}{8}$
$1''$—S	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{3}{16}$
$1\frac{1}{16}''$—R	7	$\frac{1}{8}$	2 @ $\frac{1}{8}$	3 @ $\frac{3}{16}$
$1\frac{1}{16}''$—S	7	$\frac{1}{8}$	2 @ $\frac{1}{6}$	3 @ $\frac{3}{16}$
$1\frac{1}{8}''$—R	7	$\frac{1}{8}$	2 @ $\frac{1}{6}$	3 @ $\frac{3}{16}$
$1\frac{1}{8}''$—S	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{3}{16}$
$1\frac{3}{16}''$—R	7	$\frac{1}{8}$	2 @ $\frac{3}{16}$	3 @ $\frac{3}{16}$
$1\frac{3}{16}''$—S	7	$\frac{1}{8}$	2 @ $\frac{7}{32}$	3 @ $\frac{3}{16}$

adhesive. Each class or type is divided into several different standard grades and the rules for these grades are known as "Douglas Fir Plywood, Commercial Standard CS45–42." The description of each grade as set forth by the Douglas Fir Plywood Association is given in the following paragraphs.

Moisture-Resistant Grades

These grades represent the majority of production and consist of plywood with a high degree of moisture resistance where its application requires that it shall retain its original form and practically all its strength when occasionally subjected to a thorough wetting and subsequent normal drying. Veneers 1/12 in. or more shall be used in the construction of moisture-resistant type panels 1/4 in. and upward in thickness. The veneer thickness shall be measured before the panel is sanded. This type is available in the following grades:

Sound 2 Sides (SO2S). Each face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. This grade shall present a smooth surface suitable for painting.

Sound 1 Side (SO1S). The face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face shall present a smooth surface suitable for painting. The back shall present a solid surface with all knots in excess of 1 in. patched and with the following permitted: not more than six knotholes or borer holes 5/8 in. or less in greater dimension, and splits 1/8 in. or less in width and pitch pockets not in excess of 1 in. in width or 3 in. in length or that do not penetrate through veneer to glue line. There may be any number of patches and plugs in the back.

Wallboard (WB). This is a 3-ply board of 1/4 in. or 3/8 in. sanded, or 5-ply 1/2 in. sanded thickness, the face of which shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face of this grade shall present a smooth surface suitable for painting. The back shall contain knotholes or pitch

pockets, splits, and other defects in number and size that will not seriously affect the strength or serviceability of the panel and which cannot reasonably and economically be repaired to make a sound face. All wallboard panels shall be so designated by grade-marking each panel.

Sheathing (SH). This is an unsanded plywood made only in the following sizes: thicknesses 5/16 in. and 3/8 in. 3-ply; 1/2 in. and 5/8 in. 3- or 5-ply; widths 36 in. and 48 in.; length 96 in. The face shall present a solid surface except that the following will be permitted: (*a*) not more than ten knotholes none of which shall exceed 1 1/2 in. with not more than five exceeding 3/4 in. in greatest dimension; (*b*) no group of knotholes within any 12-in. diameter circle shall have an aggregate greatest dimension of more than 3 in.; (*c*) no splits wider than 1/8 in., nor any type of borer holes longer than 1 in., nor open pitch pockets more than 1 in. wide. There may be any number of patches and plugs in the face, but the face may not be of such quality that, if sanded, it will pass for a Wallboard face. No belt sanding is permissible. The back shall be at least equal in quality to a Wallboard grade back. No tape shall be permitted in the glue line. All sheathing panels shall be scored or marked for nailing to conform to standard spacing of lumber studding.

Industrial (Unsanded). Faces of panels shall be free from knotholes, and any type of borer holes more than 5/8 in. in greatest dimension and open pitch pockets more than 1 in. wide. Tight knots, checks, plugs, patches and shims shall be admitted in either face. Core and crossbands shall be of firm stock but shall contain no knotholes greater than 1 1/4 in. in any dimension.

Concrete-Form Plywood. Concrete-form plywood shall be built up of three or five thicknesses of veneer, of which the two outside plies are at least 1/8 in. thick before sanding, except for plywood 1/4 in. in thickness. An occasional knothole is permissible in the center or core of 5-ply panels only, but no knotholes are permitted in cross banding. Appearance of faces shall be similar to that of "Sound 2 Sides" grade. The bonding agent used shall be especially prepared for this purpose and shall be very highly water-resistant. All concrete-form plywood shall be so designated by grade-marking each panel on the face. Concrete-form plywood shall be edge-sealed and shall have the faces mill-oiled unless the order specifically states not to oil.

Exterior Grades

These grades represent the ultimate in moisture resistance, a plywood that will retain its original form and strength when repeatedly wet and dried and otherwise subjected to the elements, and suitable for permanent exterior use. It shall be free from both core gaps and core voids

that impair the strength or serviceability of the panel. Only a resin-impregnated tape shall be permitted in the glue line. No veneer thicker than $5/16$ in. shall be used. All exterior panels shall be so designated by a distinctive symbol "Ext." branded or stamped on the edge of each panel. This type is available in the following grades:

Good 2 Sides Exterior (G2S-Ext.). Each face shall be of a single piece of smoothly cut veneer of 100 per cent heartwood, free from knots, splits, checks, pitch pockets, and other open defects. The faces shall be a yellow or pinkish color without stain. Shims that occur only at the ends of panels and inconspicuous well-matched small patches not to exceed $3/8$ in. in width by $2\frac{1}{2}$ in. in length shall be admitted. This grade is recommended for uses where a light stain or natural finish is desired.

Good 1 Side Exterior (G1S-Ext.). The face shall be equal to that described under "Good 2 Sides Exterior" grade, while the back shall be equal to the "Sound 2 Sides Exterior" grade.

Sound 2 Sides Exterior (SO2S-Ext.). Each face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, checks, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. This grade shall present a smooth surface suitable for painting.

Sound 1 Side Exterior (SO1S-Ext.). The face shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, it shall be well joined and reasonably matched for grain and color at the joints. It shall be free from knots, splits, pitch pockets, and other open defects. Streaks, discolorations, sapwood, shims, and neatly made patches shall be admitted. The face on this grade shall present a smooth surface suitable for painting. The back shall contain knotholes not larger than 1 in., splits not wider than $3/16$ in., and other defects in number and size that will not impair the serviceability of the panel and that cannot be reasonably and economically repaired to make a sound face.

Sheathing Exterior (SH-Ext.). An unsanded panel, the face of which shall present a solid surface except that the following will be permitted: (*a*) not more than six knotholes $3/8$ in. or less in greatest dimensions; (*b*) splits $1/16$ in. or less in width; (*c*) one or two strips of paper tape. There may be any number of patches and plugs in the face but the face may not be of such quality that, if sanded, it will pass for "Sound 1 Side Exterior" grade. No belt sanding is permissible. The back shall be the same as the back described under "Sound 1 Side Exterior" grade.

Industrial Exterior. Shall have two solid faces made of one or more pieces. All open defects shall be repaired, except small pitch pockets and

tight splits which are $\frac{1}{16}$ in. or under in width. All knotholes in the face veneer shall be patched. Panels in this grade shall be lightly "touch" sanded on both sides to remove dry tape, surplus glue, etc., but the tolerance of $\frac{1}{32}$ in., as allowed for unsanded panels, shall apply.

Concrete-Form Exterior. Shall be the same as "Sound 2 Sides Exterior" except that faces shall be $\frac{1}{8}$ in. thick before sanding. It is made only in $\frac{5}{8}$ in. and $\frac{3}{4}$ in. thicknesses. All concrete-form plywood shall be so designated by grade-marking each panel on the face. Concrete-form plywood shall be edge-sealed and shall have the faces mill-oiled unless the order specifically states not to oil.

113. Design Stresses. The following tables on design methods and allowable stresses are taken from the Forest Products Laboratory Bulletin, *Approximate Tentative Methods of Calculating the Strength of Plywood*, by L. J. Markwardt, Principal Engineer, and A. D. Freas, Assistant Engineer.

Appropriate unit stress for Table 36 may be obtained from Table 37 by multiplying the basic values by the appropriate reduction factor, as follows:

(1) For Douglas fir plywood to be used in dry locations (moisture content 16 per cent or less), the basic stresses for extreme fiber in *bending*, *compression perpendicular to grain*, and *compression parallel to grain* may be increased by 25 per cent. (No increase for maximum horizontal shear or modulus of elasticity.)

(2) The basic stresses are for clear wood without defects. An appropriate reduction factor is to be used according to the estimated grade of material with respect to defects allowed. When defects present are estimated to reduce the strength one-fourth, multiply the basic stresses by three-fourths, etc.

EXAMPLE: What unit stress should be used for tension parallel to face grain for plywood of a three-fourths grade to be used in dry locations?

Procedure: $2000 \times 1.25 \times 0.75 = 1875$ lb. per sq. in.

For 5-ply panels in compression parallel to the grain of the face plies the unit stress should be taken as that for the next lower grade.

114. Strength Properties. In computing the tensile or compressive strength of Douglas fir plywood it is customary to consider only those plies having their grain parallel to the direction of the load. For example, consider a $\frac{1}{4}$-in. (SO2S) three-ply panel that is to be used in a dry inside location.

$$\text{Thickness of face} = \tfrac{1}{9} \text{ in.}$$

$$\text{Thickness of core} = \tfrac{1}{9} \text{ in.}$$

$$\text{Thickness of back} = \tfrac{1}{9} \text{ in.}$$

Table 36. Design Method and Allowable Stresses for Douglas Fir Plywood [1]

Property	Direction of Stress with Respect to Direction of Face Grain	Portion of Cross-Sectional Area to be Considered	Unit Stress to be Used
Tension	Parallel or perpendicular	Parallel plies [2] only	Unit stress for extreme fiber in bending.
	±45°	Full cross-sectional area	One-fourth unit stress for extreme fiber in bending.
Compression	Parallel or perpendicular	Parallel plies [2] only	Unit stress in compression parallel to grain
	±45°	Full cross-sectional area	One-third unit stress in compression parallel to grain.
Shear	Parallel or perpendicular	Full cross-sectional area	Double unit stress for horizontal shear.
	±45°	Full cross-sectional area	Four times unit stress for horizontal shear.
Shear in Plane of Plies	Parallel, perpendicular, or ±45°	Full shear area	1. Conditions without stress concentration: *A*. Joints between plies in plywood panels acting as a beam—one-half unit stress for horizontal shear. 2. Conditions with stress concentration: *A*. Symmetrical concentration. Joints in panels with stressed plywood covers, for interior joists with end headers, or without end headers if ratio of joist depth to joist width does not exceed 2—one-half unit stress for horizontal shear. *B*. Unsymmetrical concentration. Joints in I and box beams with plywood webs; and joints in panels with stressed plywood covers, for *exterior* joists with end headers, or without end headers if ratio of joist depth to joist width does not exceed 2—one-fourth unit stress for horizontal shear.
Load in Bending	Parallel or perpendicular	Bending moment $M = KSI/c$, where S = unit stress for extreme fiber in bending; I = moment of inertia computed on basis of parallel plies only, c = distance from neutral axis to outer fiber of outermost ply having its grain in the direction of the span; $K = 1.50$ for three-ply plywood having the grain of the outer plies perpendicular to the span; $K = 0.85$ for all other plywood.	Unit stress for extreme fiber in bending.
Deflection in Bending	Parallel or perpendicular	Deflection may be calculated by the usual formulas, taking as the moment of inertia that of the parallel plies plus 1/20 that of the perpendicular plies. (When face plies are parallel, the calculation may be simplified, with but little error, by taking the moment of inertia as that of the parallel plies only.)	Unit value for modulus of elasticity.
Deformation in Tension or Compression	Parallel or perpendicular	Parallel plies [2] only	Unit value for modulus of elasticity.
Bearing at Right angles to Plane of Plywood		Loaded area	Unit stress in compression perpendicular to grain.

[1] The suggested simplified methods of calculation in this table apply reasonably well with usual plywood types under ordinary conditions of service. It is recognized, however, that they are not entirely valid for all types of plywood and plywood constructions, or for all spans and span-depth ratios.

[2] By "parallel plies" is meant those plies whose grain direction is parallel to the direction of principal stress.

Table 37. Basic Stresses for Douglas Fir

	Pounds per Square Inch
Extreme fiber in bending	2,000
Compression perpendicular to grain	325
Compression parallel to grain	1,466
Maximum horizontal shear	120
Modulus of elasticity	1,600,000

Table 38. Example of Unit Stresses for Douglas Fir Plywood of Three-Fourths Grade to be Used in a Dry Inside Location

Property	Direction of Stress with Respect to Face Grain	Unit Stress Pounds per Square Inch
Tension	Parallel or perpendicular	1,875
Tension	±45°	470
Compression	Parallel or perpendicular	1,375
Compression	±45°	460
Shear	Parallel or perpendicular	180
Shear	±45°	360
Shear in plane of plies		22 [1]
Bending	Parallel or perpendicular	1,875
Modulus of elasticity	Parallel or perpendicular	1,600,000
Compression perpendicular to grain (bearing)		400

[1] Applies to shear stresses, such as in I and box beams, where the stress is concentrated at the inner edge of the flange. For other conditions as outlined in Table 36, different values would be found as calculated by the appropriate factor.

Tension, allowable load per inch of width

T_L,[1] grain of face plies parallel to load =

$$2 \times \tfrac{1}{9} \times 2190 = 487 \text{ lb.}$$

T_T,[1] grain of face plies perpendicular to load =

$$1 \times \tfrac{1}{9} \times 2190 = 243 \text{ lb.}$$

Compression, allowable load per inch of width

$$C_L = 2 \times \tfrac{1}{9} \times 1605 = 356 \text{ lb.}$$

$$C_T = 1 \times \tfrac{1}{9} \times 1605 = 178 \text{ lb.}$$

[1] Subscripts L and T refer to the direction of the grain of the face plies. L denotes the face grains parallel to the applied load or parallel to the span in bending. T is the opposite condition with the face grains perpendicular to the applied load.

Table 39. Working Stresses for Douglas Fir Plywood, Damp or Wet Location

[Moisture content of the plywood under load may exceed 16 per cent. Technical Data on Plywood, Douglas Fir Plywood Association, 1942]

	(These two exterior grades available only on special mill order)		$87\frac{1}{2}\%$ SO2S	80% Plywall or SO1S	75% Sheathing
	Clear 100% G2S	95% G1S			
Extreme fiber in bending	2,000	1,900	1,750	1,600	1,500
Compression perpendicular to grain	325	325	325	325	325
Compression parallel to grain	1,466	1,390	1,285	1,170	1,100
Maximum horizontal shear	120	114	105	96	90
Modulus of elasticity	1,600,000	1,600,000	1,600,000	1,600,000	1,600,000

Table 40. Working Stresses for Douglas Fir Plywood, Dry Location

[Moisture content does not exceed 16 per cent. Technical Data on Plywood, Douglas Fir Plywood Association, 1942]

	Clear 100% G2S	95% G1S	$87\frac{1}{2}\%$ SO2S	80% Plywall or SO1S	75% or Plyscord Sheathing
Extreme fiber in bending	2,500	2,375	2,190	2,000	1,875
Compression perpendicular to grain	405	405	405	405	405
Compression parallel to grain	1,830	1,735	1,605	1,460	1,375
Maximum horizontal shear	120	114	105	96	90
Modulus of elasticity	1,600,000	1,600,000	1,600,000	1,600,000	1,600,000

Moment of inertia, for a strip 1 inch wide

$$I_L = \tfrac{1}{12}(\tfrac{1}{4})^3 - (\tfrac{1}{9})^3 = 0.00118$$

$$I_T = \tfrac{1}{12}(\tfrac{1}{9})^3 = 0.00017$$

Bending, allowable bending moment per inch of width

$$M_L = \frac{0.85SI}{c} = \frac{0.85 \times 2190 \times 0.00118}{\frac{1}{8}} = 17.6 \text{ in.-lb.}$$

$$M_T = \frac{1.5SI}{c} = \frac{1.5 \times 2190 \times 0.00017}{\frac{1}{18}} = 10.05 \text{ in.-lb.}$$

Deflection, allowable deflection for a strip 1 inch wide with a load of 40 lb. per sq. ft. over a span of 12 in.

$$Y_L = \frac{5}{384} \times \frac{40}{12} \times \frac{12 \times 12 \times 12}{1{,}600{,}000 \times 0.00118} = 0.0396 \text{ in.}$$

$$Y_T = \frac{5}{384} \times \frac{40}{12} \times \frac{12 \times 12 \times 12}{1{,}600{,}000\left(0.00017 + \dfrac{0.00118}{20}\right)} = 0.264 \text{ in.}$$

115. Gusset Plates. Plywood panels thicker than those customarily manufactured offer a convenient means of forming timber joints when bolts are used alone or with timber connectors. Several investigators have conducted tests on Douglas fir plywood to ascertain the safe working loads of connectors when plywood is used in wood to wood joints.[1] Although these tests have not covered all possible joint assemblies, certain conclusions can be made with respect to the design loads for connectors when used with plywood. The minimum recommended thicknesses of Douglas fir plywood should be the same as those recommended for Douglas fir lumber and the standard design loads for connectors should be taken as the same as those for Douglas fir when the load is at an angle of 60° to the grain. These recommendations are based on the results of tests by Prof. Williams and are fairly well substantiated by the tests conducted on 2½-in. split rings by Messrs. Benkert, Hanrahan, and Smith. They recommend, however, that the standard edge distance be increased for plywood. A minimum edge distance for the 2½ rings should be 3 in. while the minimum spacing and end distance should be 5 in.

[1] "Laboratory Tests on Structural Plywood," by H. A. Williams, *Eng. News-Record,* June 16, 1938. "Plywood for Gusset Plates," by H. N. Benkert, F. J. Hanrahan, and L. W. Smith, *Eng. News-Record,* June 23, 1938.

116. Plywood Girders. The use of plywood for the webs of large wooden **I**-beams is a comparatively recent development. The usual construction is to use a plywood web $\frac{3}{8}$ to $\frac{3}{4}$ in. thick and either solid or glued laminated flanges and stiffeners. The flanges and stiffeners may be glued or bolted to the web. Sufficient test data are lacking to provide an exact theory for use in design but the following method is suggested as that likely to give results well within a safe limit.

Consider the girder as shown in Figure 99 and determine the safe uniform load per foot that it is capable of carrying. The web is $\frac{3}{8}$ in. 3-ply Douglas fir plywood, with the face veneers $\frac{1}{8}$ in. thick and the core

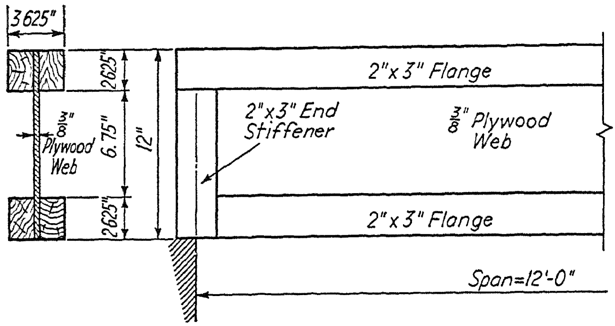


FIG. 99. Plywood girder.

$\frac{3}{16}$ in. The flanges and stiffeners are to be Douglas fir glued to the web with casein glue.

Design Stresses, Plywood with Face Grains in Direction of Span:

Extreme fiber in bending = 2500 lb. per sq. in.
Compression perpendicular to grain = 405 lb. per sq. in.
Compression parallel to grain = 1830 lb. per sq. in.
Maximum horizontal shear = 240 lb. per sq. in.
Shear in plane of plies = 30 lb. per sq. in.
Modulus of elasticity = 1,600,000 lb. per sq. in.

Design Stresses, Solid Wood:

Extreme fiber in bending = 1200 lb. per sq. in.
Compression perpendicular to grain = 325 lb. per sq. in.
Compression parallel to grain = 880 lb. per sq. in.
Maximum horizontal shear = 100 lb. per sq. in.
Modulus of elasticity = 1,600,000 lb. per sq. in.

In determining the load capacity of a plywood girder there are three main things to consider. These are (1) the flanges must resist the

moment due to the load, (2) the glue joint between the web and the flange must be sufficient to resist the horizontal shear along this line, (3) the web must be stiffened to resist the buckling loads caused by the vertical loads and reactions.

Moment of inertia and statical moment of cross-sectional area about neutral axis: In computing I and Q only the plies having their grain parallel to the span are considered.

$$I = \frac{2 \times 3.25 \times 2.625^3}{12} + 2 \times 3.25 \times 2.625 \times 4.688^2 + \frac{2}{8} \times \frac{12^3}{12}$$

$$= 421.75 \text{ in.}^4$$

$$Q = 3.25 \times 2.625 \times 4.688 + \tfrac{2}{8} \times 6 \times 3 = 44.5 \text{ in.}^3$$

Tension flange:

$$f = \frac{Mc}{I} \qquad M = \frac{fI}{c} = \frac{1200 \times 421.75}{6} = 84{,}350 \text{ in.-lb.}$$

$$\frac{w \times 12 \times 12 \times 12}{8} = 84{,}350 \qquad w = 390 \text{ lb. per ft.}$$

Compression flange: The allowable load will be governed by the compression flange rather than by the tension flange because the unit stress in compression is less than that in tension. Moreover, the unit stress in compression must be multiplied by the appropriate form factor.

$$F = 0.58 + 0.42\left(0.264\,\frac{(3.625 - 0.375)}{3.625} + \frac{0.375}{3.625}\right) = 0.72$$

$$w = \frac{880 \times 0.72 \times 390}{1200} = 206 \text{ lb. per ft.}$$

Shear between flange and web:

$$v = \frac{VQ}{Id_1} \qquad V = \text{vertical shear} = \frac{206 \times 12}{2} = 1236 \text{ lb.}$$

$$d_1 = \text{length of contact} = 2.625 \times 2 = 5.25$$

$$v = \frac{1236 \times 44.5}{421.75 \times 5.25} = 25 \text{ lb. per sq. in.}$$

The allowable unit stress is 30 lb. per sq. in.

End stiffeners: The end stiffeners prevent the web from buckling under the concentrated end reaction and should be designed to carry the entire load without aid from the web. The area in bearing between the end stiffeners and the top flange must be great enough that the unit stress in compression perpendicular to the grain is not exceeded.

$$V = \frac{206 \times 12}{2} = 1236 \text{ lb.}$$

Unit stress in compression parallel to grain $= \dfrac{1236}{2 \times 1.625 \times 1.312} =$ 290 lb. per sq. in.

Allowable unit stress $=$ 880 lb. per sq. in.

Unit stress in bearing perpendicular to grain of top flange $=$

$\dfrac{1236}{2 \times 1.625 \times 2.625} =$ 145 lb. per sq. in.

Allowable unit stress $=$ 325 lb. per sq. in.

Horizontal shear: Based on a load of 206 lb. per ft., the vertical shear $=$1236 lb.

The unit shearing stress in the plywood web $= \dfrac{1236 \times 44.5}{421.75 \times \frac{3}{8}} =$ 348 lb. per sq. in.

The maximum horizontal shear is 240 lb. per sq. in. and the allowable load must be reduced to 142 lb. per ft.

If the face plies had their grain at an angle of 45° to the axis of the beam and the grain of the core was perpendicular to the grain of the face plies, the allowable unit horizontal shear would be 480 lb. per sq. in.

Web buckling: At any point in the web the horizontal and vertical unit shears are equal and produce an equal resultant in compression which acts at an angle of 45° to the edge of the web, providing it is assumed that the web does not resist any part of the stresses due to bending. This compressive stress must be kept small enough to prevent the web from buckling or stiffeners must be added to reinforce the web.

The unit shearing stress at which buckling will occur may be expressed by the following formula:[1]

$$S_{cr} = \frac{\pi^2 E t^2 K}{12 b^2 (1 - V^2)}$$

[1] *Strength of Materials*, Part 2, by S. Timoshenko.

E = modulus of elasticity at 45°.
t = thickness of web.
b = smallest dimension.
V = Poisson's ratio.
K = a constant depending upon the equation $\left(\frac{a}{nb} + \frac{nb}{a}\right)^2$
a = largest dimension.
n = an integer representing the number of waves into which the panel will buckle.

Table 41. Values of *K* in Terms of *a* and *b*

a/b	1	1.2	1.4	1.5	1.6	1.8	2.0	2.5	3	∞
K	9.42	8	7.3	7.1	7.0	6.8	6.6	6.3	6.1	5.4

The value of Poisson's ratio for most plywood is not well known, but for practical purposes it may be taken as zero. We do know that it will be less than unity, and by assuming a value of zero, a factor of safety is introduced. The modulus of elasticity for Douglas fir plywood at 45° to the grain may be taken as 260,000.

With no stiffeners $a = 141.375$ and $b = 6.75$

$$a/b = \frac{141.375}{6.75} = 21 \qquad K = 5.4$$

$$S_{cr} = \frac{3.14^2 \times 260{,}000 \times 0.375^2 \times 5.4}{12 \times 6.75^2} = 3570 \text{ lb. per sq. in.}$$

As the above shearing stress which will cause the web to buckle is much greater than the allowable horizontal shear, no stiffeners other than the one at each end are required.

If the grain of the plies makes an angle of 45° with the axis of the beam, the modulus of elasticity is taken parallel to the face plies and becomes 1,600,000 lb. per sq. in. In this case the shearing stress required to cause buckling would be increased considerably.

A more accurate method [1] of determining the critical shearing stress is to consider the elastic constants of the plywood and determine the value of the constants D_1, D_2, and D_3 for use in the following formulas. For a simply supported rectangular plate

$$\theta = \frac{\sqrt{D_1 D_2}}{D_3} \qquad B = \frac{b}{a}\sqrt[4]{\frac{D_1}{D_2}}$$

[1] *Theory of Elastic Stability*, S. Timoshenko, 1936.

and the critical value of the shearing force when $\theta > 1$ may be obtained from the formula

$$F_{cr} = \frac{4K\sqrt[4]{D_1 D_2^3}}{b^2}$$

K is a factor depending on θ and B, which can be taken from Chart 16. This chart is the same as that given in, "The Critical Shear Load of Rectangular Plates," by E. Seydel, Tech. Memo. 705, N.A.C.A.

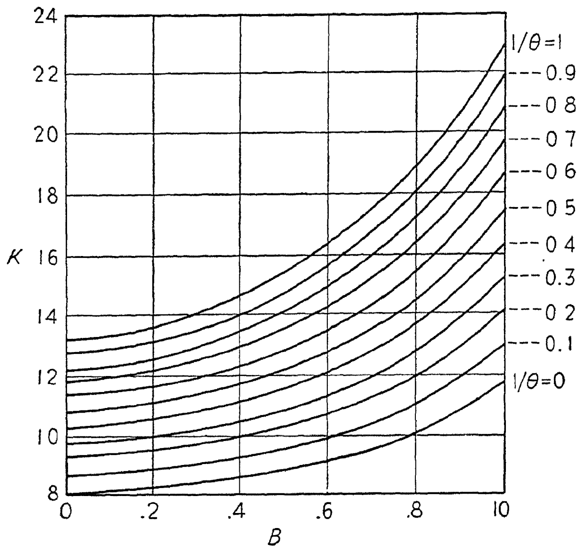


CHART 16. Values of K for use in the formula for determining the critical buckling stress due to shearing stresses in plywood plates.

When $\theta < 1$ the critical value of the shearing force may be found from the formula

$$F_{cr} = \frac{4K\sqrt{D_2 D_3}}{b^2}$$

Table 42. Values of K for an Infinitely Long Plate

θ	0	0.2	0.5	1.0
K	11.7	11.8	12.2	13.17

The above equations can be used for long plates with clamped edges when K is taken from Table 43.

Table 43. Values of K for an Infinitely Long Plate with Clamped Edges

θ	0	0.2	0.5	1	2	3	5	10	20	40	∞
K	18.6	18.9	19.9	22.15	18.8	17.6	16.6	15.9	15.5	15.3	15.1

For plywood with flat-grained veneers [1]

$$D_1 = \frac{E_1 h^3}{12V} \qquad D_2 = \frac{E_2 h^3}{12V} \qquad D_3 = \frac{E_L V_{TL} h^3}{12V} + \frac{G_{TL} h^3}{6}$$

E_1 = modulus of elasticity of the entire panel taken in a direction.
E_2 = modulus of elasticity of the entire panel taken in b direction.
h = thickness of panel.
$V = 1 - V_{LT}V_{TL}$, where V_{LT} and V_{TL} are Poisson's ratios. Their product = 0.01.
E_L = modulus of elasticity along the grain, 1,600,000.
V_{TL} = Poisson's ratio for extension along the grain and contraction tangent to growth rings = 0.019.
G_{TL} = modulus of rigidity in shear = $E_L/16$.
E_T = modulus of rigidity across the grain = $1{,}600{,}000 \times 0.045 = 72{,}000$
a = largest dimension.
b = smallest dimension.

The exact modulus of elasticity of plywood is the summation of the product of the moment of inertia of each ply about the center of the panel and its modulus of elasticity, divided by the moment of inertia of the entire cross section.

For a 3-ply panel with all the plies of the same thickness, the modulus of elasticity can be reduced to a simple form in terms of E_L and E_T.

Let T be the total thickness

$$E_1 = \frac{\left[\frac{1}{12} \times 2 \times \left(\frac{T}{3}\right)^3 + 2 \times \frac{T}{3} \times \left(\frac{T}{3}\right)^2\right] E_L + \left[\frac{1}{12} \times \left(\frac{T}{3}\right)^3\right] E_T}{\frac{T^3}{12}}$$

$$E_1 = \tfrac{1}{27}(26E_L + E_T)$$

$$E_2 = \frac{\left[\frac{1}{12} \times 2 \times \left(\frac{T}{3}\right)^3 + 2 \times \frac{T}{3} \times \left(\frac{T}{3}\right)^2\right] E_T + \left[\frac{1}{12} \times \left(\frac{T}{3}\right)^3\right] E_L}{\frac{T^3}{12}}$$

$$E_2 = \tfrac{1}{27}(E_L + 26E_T)$$

[1] For the derivation of the constants D_1, D_2, and D_3 see Article 122.

For the 3/8-in. 3-ply web under consideration the plies are not all of the same thickness. The face and back are 1/8 in. and the core is 3/16 in. This makes a total thickness of 7/16 in.

$$E_1 = 1{,}485{,}000 \qquad D_1 = 10{,}410$$

$$E_2 = 193{,}000 \qquad D_2 = 1{,}358$$

$$D_3 = 1{,}273$$

$$\theta = \frac{\sqrt{D_1 D_2}}{D_3} = 2.945 \qquad B = \frac{b}{a}\sqrt[4]{\frac{D_1}{D_2}} = 0.08$$

From Chart 16 $K = 9.8$

$$F_{cr} = \frac{4K\sqrt[4]{D_1 D_2^3}}{b^2} = \frac{4 \times 9.8\sqrt[4]{10{,}410 \times 1358^3}}{6.75 \times 6.75} = 1940$$

$$S_{cr} = \frac{1940 \times 16}{7} = 4430 \text{ lb. per sq. in.}$$

The load-carrying capacity of the girder is limited by the working stress in horizontal shear.

AIRCRAFT PLYWOODS

117. General. Plywood for use in aircraft is manufactured under very exacting conditions, and only a limited number of species are suitable for its manufacture. However, the success with which plywood has been used in aircraft construction demonstrates the ability of manufacturers to produce it with the precision commensurate with design methods. This ability may be attributed to the knowledge of the characteristics and strength properties of the various species, and the methods of handling veneers throughout the entire manufacturing process.

The chief advantages of plywood for aircraft construction are its high ratio of strength to weight, the ease with which it may be repaired, its relatively low cost, and its extreme adaptability for a number of parts.

Plywood used in aircraft construction is subjected to a combination of stresses that are seldom found, or designed for, in large engineering structures. Under combined loading the stress is not proportional to the load, and for this reason airplane parts are designed on the basis of ultimate stresses.

118. Species. The Army-Navy aeronautical specifications, AN-NN-P-511a, contain the species of wood which may be used in plywood construction. The species are divided into three groups as shown in Table 44.

Table 44. Permissible Species of Wood for Aircraft Plywood Construction

Group 1	Group 2	Group 3
Beech, American	Birch, Alaska, white, and paper	Basswood, American
Birch, sweet and yellow	Mahogany, African	Poplar, yellow
Maple, hard	Mahogany, Central American	Cedar, Port Orford
	Maple, soft	Spruce, Sitka, red, and white (quarter-sliced)
	Gum, red and tupelo	
	Walnut, black	
	Douglas fir (quarter-sliced)	

Group I contains species with high strength properties, hardness, and resistance to abrasion. The species in Groups 1 and 2 take the best finish and are best suited for the manufacture of curved parts. Group 3 woods are used primarily for core stock, but they do have high bending and buckling resistance on the basis of the strength-weight ratio.

Beech, American: Beech ranks favorably with sweet and yellow birch and hard maple in strength properties. However, it is slightly heavier and is not readily available in the high grades.

Birch, sweet and yellow: Sweet and yellow birch have the same general properties. They are hard and stiff and rather heavy. However, their fine texture makes them capable of taking a high finish. They are used primarily for propellers and veneer for plywood.

Maple, hard: Hard maple is dense, hard, and stiff and has a fine even texture. It is superior in hardness to all other species used in aircraft. It is used for propellers, veneer for plywood, and bearing blocks.

Mahogany: African mahogany is lighter than mahogany from Central America, but in most strength properties the two species are very similar.

Maple, soft: Soft maple is much lower in weight and strength than hard maple. However, it is suitable for plywood where a semi-hard species is required.

Gum: Red gum is heavier than spruce and has higher strength properties, with the exception of stiffness. It has a tendency to warp, but through careful manufacture it is a good species for plywood.

Tupelo gum is considerably heavier than spruce, but it is lower in stiffness. It can be used in plywood but is not adaptable for other aircraft uses.

WALNUT, BLACK: Black walnut is probably the best wood for propeller construction. It has good hardness to resist wear and has the ability to retain its shape under varying moisture conditions. Its fine texture and finishing qualities make it ideal for use in instrument panels and cabinets.

DOUGLAS FIR (COAST REGION): Douglas fir from the West Coast is considerably heavier than spruce and has strength properties exceeding those of spruce. Because of its tendency to splinter, considerably more care is necessary in the manufacture of Douglas fir than most of the other species. Quarter-sliced Douglas fir is suitable for aircraft plywood.

BASSWOOD, AMERICAN: Basswood is a light weight wood, but it is low in most strength properties. However, it works easily and can be nailed without splitting. It is a desirable wood for use in aircraft as veneer for plywood, wing ribs, and webs.

POPLAR, YELLOW: Yellow poplar is not much heavier than spruce, but some of its strength properties are slightly less than those of spruce. It is easily worked, retains its shape, and in some respects is a good substitute for spruce. It has been used extensively in aircraft plywood.

CEDAR, PORT ORFORD: Port Orford cedar is somewhat heavier than spruce, with strength properties slightly higher. It has not been used a great deal, but it is satisfactory for plywood.

SPRUCE, SITKA, RED AND WHITE: All three species of spruce have the same strength properties. However, Sitka spruce is more abundant and is available in larger sizes. For these reasons Sitka spruce is used more often than red and white spruces. The excellent strength properties of spruce combined with a high ratio of strength to weight make it an ideal material for use as wing beams, struts, ribs, webs, landing gear, stiffeners, flooring, longérons, and veneer for plywood. Spruce is considered the standard of comparison for the suitability of other species for aircraft construction

119. Sizes. Plywood panels for aircraft construction are made in a number of thicknesses and combinations of veneers. The required veneer thickness and species for each nominal thickness are given in the specifications AN-NN-P-511a and are reproduced here.

120. Design Values for Species Used in Aircraft Plywood. Table 46 presents strength data on various species for use in aircraft design. The values are based on a moisture content of 15 per cent and a duration of stress of 3 sec. They apply only to material that meets the minimum specific gravity requirements and the limitations as to defects. The allowable defects have been set forth in Report 354 of the National Advisory Committee for Aeronautics and reprinted in Bulletin 1079 of the United States Department of Agriculture.

Table 45. Plywood Panel Construction

			Thickness of Plies											
			Group I or II Faces with Group III Inner Plies				All Group I or all Group II				All Group III			
			F&B	XB	IB	C	F&B	XB	IB	C	F&B	XB	IB	C
			Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch	Inch
1/32	3	0.031 ± 0.004		...		...	1/100[a]		...	1/64[a]				...
3/64	3	.047 ± .006		...			1/64[a]	...		1/48[a]		...	...	...
1/16	3	.063 ± .007	1/18	...		1/32	1/48		...	1/32	1/48		...	1/28
5/64	3	.078 ± .008	1/18		...	1/24	1/48		...	1/24	1/48	...	...	1/20
3/32	3	.094 ± .009	1/32	...		1/24	1/32		...	1/24	1/32	...	...	1/20
1/8	3	.125 ± .010	1/28	...		1/16	1/28		...	1/16	1/28	...	...	1/14
5/32	3	.157 ± .011	1/24	...	...	1/12	1/24		...	1/12	1/24		...	1/12
5/32	5	.157 ± .011	1/32	1/24	...	1/32	1/32	1/24		1/32	1/32	1/24	...	1/32
3/16	3	.188 ± .012	1/20	...		1/10	1/20	...	...	1/10	1/20	...	...	1/10
3/16	5	.188 ± .012	1/28	1/20	...	1/28	1/28	1/20		1/32	1/28	1/20	...	1/28
7/32	5	.219 ± .013	1/24	1/16		1/32	1/24	1/16	...	1/32	1/24	1/16	...	1/28
1/4	5	.250 ± .014	1/20	1/16	...	1/20	1/20	1/16	...	1/24	1/20	1/16	...	1/20
5/16	5	.313 ± .016	1/16	1/12	...	1/20	1/16	1/12	...	1/24	1/16	1/12	...	1/20
3/8	5	.375 ± .018	1/16	1/10	...	1/12	1/16	1/10	...	1/12	1/16	1/10	...	1/12
7/16	7	.438 ± .020	1/16	1/16	1/16	1/10	1/16	1/16	1/16	1/10	1/16	1/16	1/16	1/10
1/2	7	.500 ± .022	1/16	1/10	1/14	1/14	1/16	1/10	1/14	1/16	1/16	1/10	1/14	1/14
9/16	7	.563 ± .023	1/16	1/10	1/10	1/12	1/16	1/10	1/10	1/12	1/16	1/10	1/10	1/12
5/8	9	.625 ± .025	1/16[b]	1/16[b]	1/14	......	1/16	1/16	1/16	...	1/16	1/16	1/14	.....
				1/10	...	1/12	......	1/10	...	1/10	...	1/10	...	1/12
3/4	9	.750 ± .028	1/12[b]	1/12[b]	1/10	...	1/12	1/12	1/10	....	1/12	1/12	1/10	......
				1/10	...	1/12	...	1/10	...	1/14	...	1/10	...	1/10
7/8	11	.875 ± .031	1/12[b]	1/12[b]	1/12	......	1/12	......	1/14	...	1/12	1/12	1/12	.....
				1/10	1/12	1/12	....	1/10	1/12	1/12	......	1/10	1/12	1/10
1	11	1.000 ± .035	1/10[b]	1/10[b]	1/10	.....	1/10	1/10	1/10	......	1/10	1/10	1/10	......
				1/10	1/10	1/10	.....	1/10	1/10	1/10	.....	1/10	1/10	1/10

[a] All plies of plywood thinner than 1/16 in. are of same species.

[b] Faces and outer cross bands of plywood having 9 or more plies are of same species.

F&B = Face and back.
XB = Cross bands.
IB = Inner bands.
C = Core.

Table 46. Design Values for Species Used in Aircraft Construction

[*Wood in Aircraft Construction*, by G. W. Trayer, National Lumber Manufacturers' Association, 1930]

Species	Minimum Specific Gravity Based on Volume and Weight Oven-Dry	Static Bending				Compression Parallel to Grain		Compression Perpendicular to Grain	Shearing Strength Parallel to Grain	Hardness, Side; Load Required to Imbed 0.444 In. Ball ½ Its Diam.
		Fiber Stress Elastic Limit Lb./Sq. In.	Modulus of Rupture Lb./Sq. In.	Modulus of Elasticity 1000 Lb./Sq. In.	Work to Maximum Load In. Lb./Cu. In.	Fiber Stress at Elastic Limit Lb./Sq. In.	Maximum Crushing Strength Lb./Sq. In.	Lb./Sq. In.	Lb./Sq. In.	Lb.
Basswood	0.36	5,600	8,600	1,250	6.6	3,370	4,500	620	720	370
Beech	.60	8,200	14,200	1,440	13.5	4,880	6,500	1,670	1,300	1,060
Birch (sweet and yellow)	.58	9,500	15,500	1,780	18.2	5,480	7,300	1,590	1,300	1,100
Cedar, Port Orford	.40	7,400	11,000	1,520	8.7	4,880	6,100	1,030	760	520
Douglas fir (coast region)	.45	8,000	11,500	1,700	8.1	5,600	7,000	1,300	810	620
Gum, red	.48	7,500	11,600	1,290	10.0	4,050	5,400	1,190	1,100	650
Gum, tupelo	.47	6,700	9,300	1,090	6.3	3,980	5,300	1,400	1,080	800
Magnolia, southern	.48	6,350	10,800	1,210	11.8	3,680	4,900	1,390	1,040	920
Mahogany, African	.42	7,900	10,800	1,280	8.0	4,280	5,700	1,400	980	720
Mahogany, Central American	.46	8,800	11,600	1,260	7.3	4,880	6,500	1,760	860	790
Maple, soft	.45	5,800	8,600	985	7.6	3,520	4,700	1,190	1,000	630
Maple, hard	.60	9,500	15,000	1,600	13.7	5,620	7,500	2,170	1,520	1,270
Spruce, red, white, and Sitka	.36	6,200	9,400	1,300	7.8	4,000	5,000	840	750	440
Walnut, black	.52	10,200	15,100	1,490	11.4	5,700	7,600	1,730	1,000	990
Yellow poplar	.38	6,000	9,100	1,300	6.5	3,750	5,000	810	800	420

NOTE: The above values are for use in Aircraft Design and are based on a duration of stress of 3 sec. and a moisture content of 15%. The true modulus of elasticity in bending, E_{TR}, may be obtained by adding 10 per cent to the modulus of elasticity given above. G, the modulus of rigidity in shear, may be taken as $E_{TR}/16$.

121. Plywood Strength Properties. Wood has widely different strength properties in the various directions relative to the grain. The strength properties may be designated by the subscripts L, T, and R as shown in Figure 100.

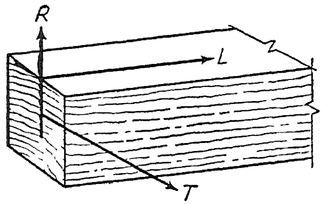


FIG. 100. Three principal axes of wood.

L = axis along the grain.
T = axis tangent to the annual growth rings.
R = axis radial to the annual growth rings.

Wood has a number of elastic constants, but for practical purposes we are interested only in those defined below.

E_L = modulus of elasticity along the grain.
E_T = modulus of elasticity tangent to the growth rings.
E_R = modulus of elasticity radial to the growth rings.
V_{LR} = Poisson's ratio, contraction along R to extension along L due to tension acting along L.
V_{RL} = Poisson's ratio, contraction along L to extension along R due to tension acting along R.
V_{TR} = Poisson's ratio, contraction along R to extension along T due to tension acting along T.
V_{RT} = Poisson's ratio, contraction along T to extension along R due to tension acting along R.
V_{LT} = Poisson's ratio, contraction along T to extension along L due to tension acting along L.
V_{TL} = Poisson's ratio, contraction along L to extension along T due to tension acting along T.
G_{LT} = modulus of rigidity in shear in LT plane.
G_{LR} = modulus of rigidity in shear in LR plane.
G_{RT} = modulus of rigidity in shear in RT plane.

The following relations exist between the elastic constants

$$V_{LT}E_T = V_{TL}E_L$$
$$V_{LR}E_R = V_{RL}E_L$$
$$V_{RT}E_T = V_{TR}E_R$$

The elastic constants have been determined for only a few species. These are given in Table 47.

The strength properties of plywood are calculated on the basis of reducing proportionally the veneer thicknesses, so that the summation of the thickness of all veneers equals the nominal plywood thickness.

STATIC BENDING

Modulus of elasticity:

The modulus of elasticity of a plywood panel is a summation of the product of the modulus of elasticity of each ply and its moment of inertia about the neutral axis divided by the moment of inertia of the geometrical cross section of the panel. The general formula for E_1 and E_2 is written:

$$E_1 = \frac{1}{I}\sum_{i=1}^{i=n} E_i I_i$$

E_1 = modulus of elasticity measured parallel to the grain of the face plies.

E_2 = modulus of elasticity measured perpendicular to the grain of the face plies.

The modulus of elasticity for the various species may be taken from Table 46. These values are in the direction of or parallel to the grain. For values across the grain multiply the modulus of elasticity along the grain by 0.045.

EXAMPLE. · Consider a $\frac{3}{16}$ in. 3-ply birch panel 1 in. wide.

$E_L = 1{,}780{,}000$ Face $= \frac{1}{10}$ in.
$E_T = 80{,}200$ Core $= \frac{1}{20}$ in.
$I = \frac{1}{12}(\frac{3}{16})^3$ Back $= \frac{1}{10}$ in.
Nominal thickness $= \frac{3}{16}$ in.

Adjusted values:

Face $= \frac{3}{64}$ in.
Core $= \frac{3}{32}$ in.
Back $= \frac{3}{64}$ in.

$$E_1 = \frac{1{,}780{,}000 \times 2[\frac{1}{12}(\frac{3}{64})^3 + \frac{3}{64}(\frac{9}{128})^2] + 80{,}200[\frac{1}{12}(\frac{3}{32})^3]}{\frac{1}{12} \times (\frac{3}{16})^3}$$

$E_1 = 1{,}567{,}000$

$$E_2 = \frac{80{,}200 \times 2[\frac{1}{12}(\frac{3}{64})^3 + \frac{3}{64}(\frac{9}{128})^2] + 1{,}780{,}000[\frac{1}{12}(\frac{3}{32})^3]}{\frac{1}{12}(\frac{3}{16})^3}$$

$E_2 = 293{,}000$

Table 47. Elastic Constants for Wood

[Report on Materials of Construction Used in Aircraft, by C. F. Jenkins, Aeronautical Research Committee (British), 1920]

		Compression			Tension			Shear			Poisson's Ratio			Constants			Shear Moduli × 10[4] (from shear tests)			Shear Moduli × 10[4] (calculated from 45° comp. test)		
	—	*L*	*R*	*T*	*L*	*R*	*T*	*LT*	*LR*	*RT*							*LT*	*LR*	*RT*	*LT*	*LR*	*RT*
Spruce: Density 27 lb. per cu. ft. Moisture 12.2%	$E \times 10^6$	1.95	0.13	0.07	...	...	...	...	...	...	V_{LR} 0 45	V_{LT} 0.539	V_{RT} 0.559	*a* 0.156	*b* 0.0897	*c* 2.027						
	El. Lt.	5,900	320	290	...	...	...	...	...	...	V_{RL} 0.03	V_{TL} 0.0194	V_{TR} 0.301	*d* 0 0648	*e* 0.09	*f* 0.0506	7.2	10 4	0 46	9 3	15.3	0.53
	Ult.	7,000	700	900	18,000	800	600	1,100	1,200	300 approx.												
Light Mahogany: Density 33 lb. per cu. ft. Moisture 13.4%	$E \times 10^6$	1.8	0.14	0.07	...	...	...	...	...	...	V_{LR} 0.31	V_{LT} 0.552	V_{RT} 0.836	*a* 0.223	*b* 0.112	*c* 1.89						
	El. Lt.	5,640	460	240	...	...	...	...	...	...	V_{RL} 0 0241	V_{TL} 0.0215	V_{TR} 0.405	*d* 0.092	*e* 0.122	*f* 0.096	6 8	8.75	2.2	7.0	9.2	2.0
	Ult.	6,500	1,920	1,000	17,800	1,080	650	1,050	1,540	500 approx												
Ash: Density 50 lb. per cu. ft. Moisture 13.6%	$E \times 10^6$	2.18	0.238	0.14	...	...	...	...	...	...	V_{LR} 0.533	V_{LT} 0.653	V_{RT} 0.656	*a* 0.352	*b* 0.206	*c* 2.47						
	El. Lt.	3,800	400	170	...	...	...	...	...	...	V_{RL} 0.0582	V_{TL} 0.0421	V_{TR} 0.386	*d* 0.214	*e* 0.284	*f* 0.148	8.9	12.4	3.6	16.0	21.8	4.8
	Ult.	7,300	2,450	2,000	25,400	2,270	1,080	2,000	2,000	600 approx.												
Walnut: Density 37 lb. per cu. ft. Moisture 11.0%	$E \times 10^6$	1.63	0.172	0 092	...	...	...	...	...	...	V_{LR} 0.495	V_{LT} 0.632	V_{RT} 0.718	*a* 0.259	*b* 0.139	*c* 1.82						
	El. Lt.	4,200	650	520	...	...	...	...	...	...	V_{RL} 0.052	V_{TL} 0.036	V_{TR} 0.367	*d* 0 140	*e* 0.190	*f* 0.107	0.1	13.9	3.4	10.2	12 5	3.2
	Ult.	8,480	1,700	1,380	24,000	1,690	1,080	2,000	2,000	500 approx.												

Stiffness factor:

The stiffness factor is the product of the modulus of elasticity and the moment of inertia.

$$E_1I = 1{,}567{,}000 \times \tfrac{1}{12}(\tfrac{3}{16})^3 = 860.0$$

$$E_2I = 293{,}000 \times \tfrac{1}{12}(\tfrac{3}{16})^3 = 160.8$$

Bending:

Values for moment at proportional limit are the bending moments that cause 1 in. plywood strips to reach their proportional limit.

M_L = moment when grain of face plies is parallel to the direction of the span.

M_T = moment when grain of face plies is perpendicular to the direction of the span.

$$M_L = \frac{E_1I \times f}{C \times E_L} \times 0.85$$

$$M_T = \frac{E_2I \times f}{C \times E_L} \times 0.90$$

E_1I = stiffness factor.

E_2I = stiffness factor.

f = fiber stress at proportional limit for solid wood from Table 46.

C = distance from neutral axis to extreme fiber of outermost longitudinal ply.

E_L = modulus of elasticity of outermost longitudinal ply.

Values for the moments at modulus of rupture are those moments which will cause the panel to fail. The formulas given above apply except that the modulus of rupture for solid wood is used for f.

EXAMPLE. $\tfrac{3}{16}$ in. 3-ply birch panel 1 in. wide.

Moment at proportional limit:

$$M_L = \frac{860 \times 9500 \times 0.85}{\tfrac{3}{32} \times 1{,}780{,}000} = 41.50$$

$$M_T = \frac{160.8 \times 9500 \times 0.90}{\tfrac{3}{64} \times 1{,}780{,}000} = 16.50$$

Moment at failure:

$$M_L = \frac{860 \times 15{,}500 \times 0.85}{\tfrac{3}{32} \times 1{,}780{,}000} = 67.80$$

$$M_T = \frac{160.8 \times 15{,}500 \times 0.90}{\tfrac{3}{64} \times 1{,}780{,}000} = 26.90$$

Tension

The total tension in the panel must be equal to the summation of the tension in the longitudinal plies and the tension in the transverse plies.

f_P = stress in panel.
T = thickness of panel.
f_L = stress in longitudinal plies.
t_L = thickness of longitudinal plies.
f_T = stress in transverse plies.
t_T = thickness of transverse plies.
Δ_L = elongation of longitudinal plies.
Δ_T = elongation of transverse plies.
E_L = modulus of elasticity along the grain.
E_T = modulus of elasticity across the grain.

$$f_P T = f_L t_L + f_T t_T$$

$$\Delta_L = \frac{f_L}{E_L} \qquad \Delta_T = \frac{f_T}{E_T}$$

$$\Delta_L = \Delta_T$$

$$\frac{f_L}{E_L} = \frac{f_T}{E_T}$$

$$f_T = \frac{E_T}{E_L} f_L$$

$$f_P T = f_L t_L + f_L \frac{E_T}{E_L} t_T$$

$$f_P = \frac{f_L t_L + f_L \frac{E_T}{E_L} t_T}{T}$$

Example. $\frac{3}{16}$ in. 3-ply birch panel.

Face grains parallel to load:

E_L = 1,780,000
E_T = 80,200
f_L = 15,500, modulus of rupture

Face = $\frac{1}{10}$ in.
Core = $\frac{1}{20}$ in.
Back = $\frac{1}{10}$ in.

Adjusted values:
T = $\frac{3}{16}$ in.
Face = $\frac{3}{64}$ in.
Core = $\frac{3}{32}$ in.
Back = $\frac{3}{64}$ in.

$$f_P = \frac{15{,}500 \times \frac{3}{32} + 15{,}500 \times 0.045 \times \frac{3}{32}}{\frac{3}{16}}$$

f_P = 8100 lb. per sq. in.

When the face grains are perpendicular to the applied load, the result is the same because the combined thickness of the face and back is the same as the core thickness for this particular panel.

When the core is of a species different from the face and back the same procedure is used.

EXAMPLE. $\frac{3}{16}$ in. 3-ply panel with birch faces and basswood core.

Birch	*Basswood*
$E_L = 1{,}780{,}000$	$E_L = 1{,}250{,}000$
$E_T = 80{,}200$	$E_T = 56{,}250$

Face grains parallel to load:

$$f_P = \frac{15{,}500 \times \frac{3}{32} + 15{,}500 \times \dfrac{56{,}250}{1{,}780{,}000} \times \frac{3}{32}}{\frac{3}{16}}$$

$$f_P = 7980 \text{ lb. per sq. in.}$$

Face grains perpendicular to load:

$$f_P = \frac{8600 \times \frac{3}{32} + 8600 \times \dfrac{80{,}200}{1{,}250{,}000} \times \frac{3}{32}}{\frac{3}{16}}$$

$$f_P = 4570 \text{ lb. per sq. in.}$$

COMPRESSION

Modulus of elasticity:

E_1 = modulus of elasticity when face grain is parallel to the direction of force.

E_2 = modulus of elasticity when face grain is perpendicular to the direction of force.

E_1 and E_2 are a summation of the product of the modulus of elasticity and the cross-sectional area of each ply divided by the total cross-sectional area. The modulus of elasticity of each ply should be taken as 10 per cent greater than the values given in Table 46.

EXAMPLE. $\frac{3}{16}$ in. 3-ply birch panel 1 in. wide.

$$E_1 = \frac{1{,}958{,}000 \times \frac{3}{32} + 88{,}000 \times \frac{3}{32}}{\frac{3}{16}} = 1023$$

$$E_2 = \frac{88{,}000 \times \frac{3}{32} + 1{,}958{,}000 \times \frac{3}{32}}{\frac{3}{16}} = 1023$$

Fiber stress at proportional limit:

F_L = fiber stress when face grain is parallel to load.
F_T = fiber stress when face grain is perpendicular to load.

$$F_L = \frac{E_1 \times f_2}{E_L}$$

$$F_T = \frac{E_2 \times f_2}{E_L}$$

f_2 = fiber stress at proportional limit for solid wood in compression parallel to grain.
E_L = modulus of elasticity along the grain.
Both values from Table 46.

$\frac{3}{16}$ in. 3-ply birch panel 1 in. wide.

$$F_L = \frac{1023 \times 5480}{1{,}780{,}000} = 3150 = F_T, \text{ for this particular panel.}$$

Maximum crushing strength:

$$F_L = \frac{E_1 \times f_3}{E_L}$$

$$F_T = \frac{E_2 \times f_3}{E_L}$$

f_3 = maximum crushing strength for solid wood in compression parallel to grain (from Table 46).

$$F_L = \frac{1023 \times 7300}{1{,}780{,}000} = 4200$$

122. Bending and Buckling of Plywood Plates. The general equation for the bending of plywood plates can be derived in the same manner as the equation for isotropic materials.[1] It is necessary, of course, to use different constants in setting up the equations, and the derivation is based on the assumption that cross sections remain plane during bending and rotate only with respect to their neutral axes.

Consider a panel subjected to uniformly distributed bending moments along its sides as shown in Figure 101.[1] The plane midway between the faces is taken as the xy plane with the z axis perpendicular to it. In Figure 102 [1] is shown an element cut from the plate.

M_x = bending moment per unit length of edges parallel to y axis.
M_y = bending moment per unit length of edges parallel to x axis.
h = total thickness.

$\frac{1}{r_x}$ = curvature of neutral section parallel to zx plane.

$\frac{1}{r_y}$ = curvature of neutral section parallel to zy plane.

[1] *Theory of Elastic Stability*, S. Timoshenko, 1936.

The unit elongation of the element *abcd* in the x and y directions at a distance z from the neutral surface may be represented as follows:

$$\epsilon_x = \frac{z}{r_x} \qquad \epsilon_y = \frac{z}{r_y}$$

From Hooke's law:

$$\epsilon_x = \frac{S_x}{E_x} - \frac{V_{yx}S_y}{E_y} \qquad \epsilon_y = \frac{S_y}{E_y} - \frac{V_{xy}S_x}{E_x}$$

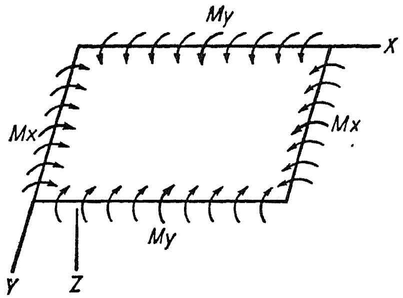


FIG. 101. Plate subjected to bending moments.

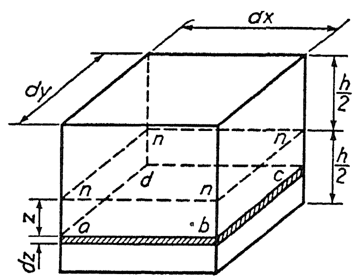


FIG. 102. Element cut from plate.

From the above equations, letting $1 - V_{yx}V_{xy} = V$

$$S_x = \frac{E_xZ}{V}\left(\frac{1}{r_x} + V_{yx}\cdot\frac{1}{r_y}\right)$$

$$S_y = \frac{E_yZ}{V}\left(\frac{1}{r_y} + V_{xy}\cdot\frac{1}{r_x}\right)$$

where S_x and S_y are the stresses in the element *abcd*.

These normal stresses produce couples which must be equal to the external moments.

$$M_xdy = \int_{-h/2}^{h/2} S_xZ\,dy\,dz$$

$$M_ydx = \int_{-h/2}^{h/2} S_yZ\,dx\,dz$$

Substituting the values for S_x and S_y and integrating

$$M_x = \frac{E_x}{V}\left(\frac{1}{r_x} + V_{yx}\cdot\frac{1}{r_y}\right)\frac{h^3}{12}$$

$$M_y = \frac{E_y}{V}\left(\frac{1}{r_y} + V_{xy}\cdot\frac{1}{r_x}\right)\frac{h^3}{12}$$

If we denote the deflection by w, the formulas for the curvature of the plate may be expressed in terms of w.

$$\frac{1}{r_x} = -\frac{d^2w}{dx^2} \qquad \frac{1}{r_y} = -\frac{d^2w}{dy^2}$$

Substituting these values in the equations for M_x and M_y, we have the following:

$$M_x = -\frac{E_xh^3}{12V}\left(\frac{d^2w}{dx^2} + V_{yx}\frac{d^2w}{dy^2}\right)$$

$$M_y = -\frac{E_yh^3}{12V}\left(\frac{d^2w}{dy^2} + V_{xy}\frac{d^2w}{dx^2}\right)$$

The twisting moments M_{xy} and M_{yx} may be derived and have the following values:

$$M_{xy} = -M_{yx} = \frac{G_{xy}h^3}{6}\frac{d^2w}{dx\,dy}$$

For a distributed lateral load acting perpendicular to the middle plane of the plate, the vertical shearing forces must be considered. These will have the following values:

$$P_x = \int_{-h/2}^{h/2} q_x\,dz \qquad P_y = \int_{-h/2}^{h/2} q_y\,dz$$

For equilibrium of the forces acting on the element it is necessary to consider the projections of all forces on the z axis and the moments of all forces with respect to the x and y axes.

Projection of forces on the z axis

$$\frac{d\,P_x}{dx}\,dx\,dy + \frac{d\,P_y}{dy}\,dy\,dx + W\,dx\,dy = 0$$

where W is the load per unit area on the face of the plate.

Simplified, the above formula becomes

$$\frac{d\,P_x}{dx} + \frac{d\,P_y}{dy} + W = 0$$

Moments of all forces acting on *abcd* with respect to the x axis

$$\frac{d\,M_{xy}}{dx} - \frac{d\,M_y}{dy} + P_y = 0$$

Moments of all forces acting on *abcd* with respect to the y axis

$$\frac{d\,M_{yx}}{dy} - \frac{d\,M_x}{dx} - P_x = 0$$

By substitution

$$\frac{d^2M_x}{dx^2} + \frac{d^2M_{yx}}{dx\,dy} + \frac{d^2M_y}{dy^2} - \frac{d^2M_{xy}}{dx\,dy} = -W$$

$$M_{yx} = -M_{xy}$$

$$\frac{d^2M_x}{dx^2} - 2\frac{d^2M_{xy}}{dx\,dy} + \frac{d^2M_y}{dy^2} = -W$$

This is the same derivation as that given by Professor Timoshenko.

Substituting the values obtained for M_x, M_y, and M_{xy} in the above equations, we obtain

$$D_1\frac{d^4w}{dx^4} + B\frac{d^4w}{dx^2dy^2} + 2K\frac{d^4w}{dx^2dy^2} + D_2\frac{d^4w}{dy^4} + C\frac{d^4w}{dy^2dx^2} = W$$

where $D_1 = \frac{E_xh^3}{12V}$ $\quad B = D_1V_{yx}$ $\quad D_2 = \frac{E_yh^3}{12V}$ $\quad C = D_2V_{xy}$

$$K = \frac{G_{xy}h^3}{6}$$

This equation can be further simplified by letting

$$D_3 = \frac{B + C + 2K}{2}$$

$$D_1\frac{d^4w}{dx^4} + 2D_3\frac{d^4w}{dx^2\,dy^2} + D_2\frac{d^4w}{dy^4} = W$$

This is the general equation for the deflection surface of a plywood plate.

For plywood made with flat-grained or rotary-cut veneers:

$$D_1 = \frac{E_1h^3}{12V} \qquad D_2 = \frac{E_2h^3}{12V}$$

$$\frac{E_xV_{yx}}{V} = \frac{E_yV_{xy}}{V} = \frac{E_LV_{TL}}{V}$$

$$D_3 = \left(\frac{E_LV_{TL}}{V}\frac{h^3}{12} + E_LV_{TL}\frac{h^3}{12} + 2G_{TL}\frac{h^3}{6}\right)\frac{1}{2}$$

$$D_3 = \frac{h^3}{12V}(E_LV_{TL} + 2G_{TL}V)$$

The critical buckling stress for a plate subjected to compression on two opposite sides may be found by substituting the above values for D_1, D_2, and D_3 in Professor Timoshenko's formula.

$$S_{cr} = \frac{\pi^2}{b^2 h}\left(D_1 \frac{b^2}{a^2} + 2D_3 + D_2 \frac{a^2}{b^2}\right)$$

$$S_{cr} = \frac{\pi^2}{b^2 h} \cdot \frac{h^3}{12V}\left(E_1 \frac{b^2}{a^2} + 2E_L V_{TL} + 4G_{TL}V + E_2 \frac{a^2}{b^2}\right)$$

$$S_{cr} = \frac{\pi^2 h^2}{12b^2 V}\left(E_1 \frac{b^2}{a^2} + 2E_L V_{TL} + 4G_{TL}V + E_2 \frac{a^2}{b^2}\right)$$

Smallest value of the critical stress is obtained when

$$\frac{a}{b} = \left(\frac{D_1}{D_2}\right)^{1/4} = \left(\frac{E_1}{E_2}\right)^{1/4}$$

$$S_{cr} = \frac{2\pi^2}{b^2 h}\left(\sqrt{D_1 D_2} + D_3\right)$$

a = unloaded side in compression and largest dimension in shear.

b = loaded side in compression and smallest dimension in shear.

E_1 = modulus of elasticity in a direction.

E_2 = modulus of elasticity in b direction.

123. Box Beams. In designing box beams with plywood webs it is necessary to multiply the fiber stress by the appropriate form factor as explained in Article 88.

Shear, actual:

The maximum shear stresses in the plywood webs may be computed by the following formula:

$$q = \frac{VQ}{It}$$

q = shear stress in pounds per square inches.

V = external shear.

Q = statical moment of area above or below neutral axis.

I = moment of inertia.

t = combined thickness of webs.

Shear, allowable:

When the grain of the veneers is alternately parallel and perpendicular to the axis of the beam, consider only those veneers having the grain parallel in computing I and Q. For 45° plywood use one-half the thickness in computing I and Q.

Allowable shear stresses are suggested as follows:

1. 2-ply or 3-ply, 45° plywood, no diaphragms. Use 4⁄3 value of shearing stress parallel to grain of solid wood.
2. Diaphragm spacing 1½ to 2½ times distance between flanges. Use 5⁄3 value of shearing stress parallel to grain of solid wood.
3. Diaphragm spacing 1½ times distance between flanges. Use 2 times value of shearing stress parallel to grain of solid wood.

When grain of plies is alternately parallel and perpendicular to the axis of the beam use 87½ per cent of the above values.

The above method of obtaining the allowable shear stresses in plywood webs was recommended by Trayer, *Wood in Aircraft Construction*, 1930.

The following formula for 45° spruce or mahogany plywood, recommended by Miller, gives somewhat higher values:

$$F_s = 960 + \frac{3140}{\sqrt[3]{C}} - 45.5\,D$$

F_s = allowable shear stress.
C = center-to-center spacing of diaphragms.
D = distance between centroids of flanges.

If C is greater than 32, use 32.

Use 87½ per cent of the above values when grain of plywood is parallel and perpendicular to axis of beam.

Glue stress between flange and web:

The stress on the glue area between the web and the flange may be calculated as follows:

$$F_G = \frac{qt'}{d}$$

F_G = shear stress on glue area.
q = maximum shear stress in plywood.
t' = thickness of one web.
d = depth of flange.

The allowable shear stress is obtained by taking ⅓ of the value for shearing stress parallel to the grain for the species having the lowest value.

PROBLEMS

1. Find the load that a ¾ in. Douglas fir plywood strip 1 in. wide can carry in tension and compression. Use grade SO2S in a dry inside location. Consider the face plies parallel to the direction of the load.

2. For the same size and grade of Douglas fir plywood as that in the previous problem find the allowable bending moment. Determine the deflection over a span of 2 ft. for the load that produces the allowable bending moment. Consider the face plies parallel to the direction of the span.

3. A 1-in. rough Douglas fir plywood panel is used as a sub-flooring over joists spaced 28 in. on center. The load is 40 lb. per sq. ft. Calculate the deflection when the face plies are parallel to the span and a three-fourths grade is used.

4. Plywood girders spaced 12 ft. apart support a uniformly distributed load of 40 lb. per sq. ft. over a span of 24 ft. Using select structural Douglas fir flanges and stiffeners and an SO2S grade of plywood for the web, design the girder.

5. A plywood girder spans 36 ft. and carries a load of 400 lb. per ft. If select structural Douglas fir is used in the flanges and stiffeners, which are attached to the web by means of split ring connectors and bolts, and a G2S grade of plywood is used in the web, what size girder is required and how many bolts and split rings are needed?

6. For a ⅝-in. nine-ply all birch panel compute the stiffness factor, the moment at the proportional limit, and the modulus of elasticity in compression. Assume a strip 1 in. wide and calculate the above properties for both directions of the face grain.

7. Compute the modulus of elasticity in static bending and the ultimate tensile strength for a 7/32-in. five-ply panel with birch faces and poplar cross bands and core.

8. A ⅜-in. five-ply all birch panel is 30 in. wide and 60 in. long. If a load in compression is applied across the 30-in. width find the critical buckling stress.

9. What is the critical buckling stress for the panel in problem 6 if the panel is 30 in. wide and 90 in. high and load in compression is applied across the 30-in. width.

10. Locate the neutral axis of the box beam shown in Figure 103 and determine the internal resisting moment and the allowable vertical shear. The webs are mahogany plywood and the flanges are spruce.

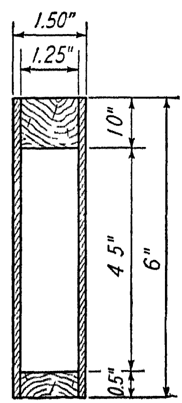


FIG. 103. Box beam.

CHAPTER IX

DECAY, WOOD-DESTROYING ORGANISMS, AND PRESERVATIVES

DECAY

124. General. Unless wood is given an approved preservative treatment or unless it contains a high percentage of naturally durable heartwood it will decay when used under high humidity conditions. Excessive dampness encourages the growth of wood-destroying fungi, which usually appear as lacelike plants on the surface of wood. These plants require food, moisture, air, and a favorable temperature in order to live, and most species of wood contain the required food supply in their cells or cell walls. Wood with a moisture content above the fiber saturation point is particularly susceptible to decay by fungi, but wood that is continuously soaked in water or continuously dry will not decay.

125. Stains and Molds. Wood-destroying fungi in the early stages of their growth produce stains or molds that are not particularly injurious to the wood provided the wood does not remain in a damp location. Little discoloration of the wood is caused by molds, and these can be easily brushed or surfaced off. Fungi cause stains which usually appear as spots or streaks on the surface of wood, the most common of which is known as blue stain. If the wood is not permitted to remain under improper moisture and temperature conditions, blue stain will not develop into a stage of decay so is not objectionable where appearance is not the governing factor.

126. Prevention of Decay. A general precaution to prevent decay in building construction is to use seasoned lumber and to provide suitable ventilation and construction features so that no condensation of moisture will take place on the wood. The roofs of weave sheds, bleacheries, and other factories where high humidities exist should be provided with an efficient moisture barrier beneath the roof rafters so that moisture will be kept from the upper part of the roof where it will condense.

Because laminated floors and built-up beams and columns of unseasoned lumber cannot dry out properly when put in place they are apt to decay.

Foundations for buildings which enclose the entire structure should have openings to provide proper ventilation under the house to prevent

decay. The usual method is to use screened openings with an area totaling about 3 per cent of the house area.

Wood posts should not bear directly on a basement floor, nor should they be embedded directly into the concrete. Instead, concrete base blocks extending above the floor surface should be used to provide the proper drainage and protection against fungus.

During construction, lumber should be properly stacked and covered to protect it from rain. The building itself may get wet, but the surface wetting of the open framing will dry off quickly.

Window and door frames should be pitched away from the building so that no moisture can accumulate, and drainage outlets and ventilation should be provided under all outside columns.

WOOD-DESTROYING ORGANISMS

127. Marine Borers. There are two main classes of marine borers—the molluscan and the crustacean. Although both groups are found in salt or brackish water, their methods of attack on wood are somewhat different. Molluscans usually enter the wood when they are very small, leaving a small entrance hole, and then grow larger as they burrow into the wood. The crustaceans attack the wood from the outside, so the destruction is more readily measured by visual inspection than that by the molluscan group.

The molluscan group includes several species of *Teredo*, *Bankia*, and *Martesia*, and the crustacean group includes *Limoria*, *Chelura*, and *Sphaeroma*.

No untreated wood is immune to the attack of marine borers, and although most borers exist only in brackish waters, some of the molluscan group have been found in fresh water in Australia, India, and parts of South America.

128. Termites. There are two types of termites, (1) the ground-inhabiting or subterranean termites and (2) the dry wood termites. Both resemble ants somewhat in size, general appearance, and living habits and for this reason are often called "white ants." The subterranean termites are found in nearly every state in this country and do most of the damage to wood structures. The dry wood termites are found along a narrow strip of territory from central California along the southern boundary of the United States to Virginia. The subterranean termites require moisture to sustain life so must have access to the ground at all times. Consequently, they build tunnels through the earth and around obstructions in order to get at their chief source of food supply, which is the cellulose in wood. The dry wood termites are

able to live in wood without contact with the ground; and although they do only a small fraction of the total destruction by termites, they are a definite menace in some regions.

Very few species of wood are capable of resisting attack by termites. California redwood and southern cypress when used above ground will offer some resistance, and the resinous heartwood from southern pine is rather resistant; but it is not available in quantities large enough to be of importance in the construction industry.

129. Protection of Wood. The only chemical method of protecting wood from attack by marine borers is to use a heavy treatment of an approved preservative. The preservative should penetrate into the piece as deeply as possible, and the wood should absorb a high percentage of the preservative.

When wood is used in contact with the ground, preservative treatments are also recommended. In many cases it is impossible or impracticable to treat all wood used above the ground that might be subject to termite attack. The best protection in this case is to construct the building in such a manner that the subterranean termites cannot gain access to it. A number of precautions or methods of construction to avoid termite attack are enumerated below.[1]

1. Construct foundations of brick, stone, concrete, or fully treated lumber piling. When a mortar is used, cement mortar is preferable. If there is a basement, it should be floored with concrete.

2. Posts that support the first floor beams should not rest on the ground or wood blocking unless it is thoroughly treated. They should be supported by concrete piers extending above the basement floor.

3. Avoid dampness, warmth, and darkness by providing ventilation under the building.

4. Construct concrete floors on a gravel base to prevent dampness and cracking.

5. Do not leave stakes or forms embedded in the concrete or next to concrete foundations because they will afford an easy means for the termites to enter the building.

6. Remove all debris from under the building, whether above or below the ground.

7. Foundations under the building should extend 12 to 18 in. above the ground. The exterior grade line may be 6 in. from the closest wood if the foundation can be readily inspected for termite tubes.

8. Place termite shields, which are sheets of galvanized iron, in the surface of the masonry and let them project horizontally for 2 in. and

[1] "Preventing Damage by Termites or White Ants," by T. E. Snyder, U. S. Dept. Agr. Bull. 1472.

then downward at an angle of 45° for 2 in. These make it impossible for termites to build their tubes up the side of the foundation and into the wood. Place circular metal shields projecting 3 in. on all sides of pipes under the building.

Paint is a good protection against dry wood termites, and all exposed portions of a building in a locality where dry wood termites are prevalent should be adequately painted. Interior unpainted portions may be protected by placing fine screens over all openings.

PRESERVATIVES

130. General. The use of suitable preservatives can greatly extend the life of wood that is used under conditions favoring decay or attack by insects or marine borers. Some preservatives are more effective than others, and the adoption of any particular type depends upon the ultimate use of the lumber. In general, a preservative is a chemical of high toxicity. An effective preservative must remain chemically stable, have a high degree of permanence, possess good penetrating properties, and be harmless to wood as well as metal. Other requirements depending upon the use for which the lumber is intended may be cleanness, cheapness, paintability, freedom from odor, and fire resistance.

Wood preservatives may be divided into three general classes: (1) toxic oils, which are relatively insoluble in water, (2) salts injected into the wood in the form of water solutions, and (3) small percentages of a toxic material combined with a solvent other than water.

131. Creosote. There are many types of creosote preservatives, but of all of them coal-tar creosote is the most important and most generally useful. It is a black or dark brown oil that is made by the distillation of coal tar. It has high toxicity, is relatively insoluble in water, is easy to apply, and is generally available at low cost. As yet no better preservative has been developed for treating timbers for general outdoor use, but for some purposes coal-tar creosote has disadvantages. It has an objectionable odor, and when freshly applied it will ignite easily. Moreover, creosoted wood cannot be painted and is not clean to handle.

Wood-tar creosote has not been used extensively enough to compare it with coal-tar creosote, but it is generally believed to be effective when properly prepared and injected.

Water-gas-tar creosote is made by the distillation of water-gas-tar and is, therefore, a petroleum product. It is not as toxic or as effective as coal-tar creosote but is considered a good preservative.

Mixtures of coal-tar with coal-tar creosote are often used for the treatment of ties and to some extent for other purposes. The addition of the

coal-tar reduces the cost of the preservative; but the solutions do not penetrate as easily as the straight coal-tar creosote, and they make the timber considerably dirtier to handle.

The addition of petroleum to coal-tar creosote makes a solution that is used extensively for the treatment of railroad ties. This solution does not penetrate as readily and is less toxic than the straight coal-tar creosote, but actual records of its use have demonstrated that it has similar decay resistance.

132. Water-Soluble Salts. A number of water-soluble salts, including zinc chloride, chromated zinc chloride, sodium fluoride, arsenic, copper sulphate, and mercuric chloride, are used as wood preservatives. Of all these, zinc chloride has been most widely used in the United States. Its chief advantages are cleanliness, paintability, lack of fire hazard, lack of odor, general availability, and relatively low cost. Its main disadvantage, which is common to all water-soluble salts, is its tendency to leach out of the wood when it is in contact with water or soil.

Chromated zinc chloride is rapidly taking the place of zinc chloride as the leading water-soluble salt preservative. It contains 18½ per cent sodium dichromate and the remaining percentage is the standard zinc chloride. Tests indicate that it is superior to zinc chloride.

Sodium fluoride has been used to some extent in this country for treating mine timbers and ties, and when mixed with other materials it has been used for treating structural timbers. It seems to be a good preservative, but it has not been used extensively enough for comparison with other salt treatments.

Arsenic has been used for several years in treating poles and ties. Used either alone or mixed with other materials, it has good preservative qualities. However, no definite long-time records are available as to its effectiveness.

Copper sulphate has not been used to any extent in the United States. It tends to corrode steel, and its use requires special treating equipment.

133. Toxic Materials Dissolved in Solvents Other Than Water. In the past several years a number of preservatives have been developed that may be applied without heat. Usually all that is necessary is to soak the wood in the preservative. These preservatives are colorless, odorless, clean, and paintable. They do not require a great deal of time to dry, and they will not swell the wood to any appreciable extent. They are used primarily for the treatment of window frames and sash, doors, flooring, and millwork.

Preservatives of this type that are commercially available contain a toxic chemical usually in the form of pentachlorphenol, or tetrachlorphenol and a volatile solvent such as solvent naphtha. These preserva-

tives have demonstrated their effectiveness as a treatment for window sash and frames, and they may prove suitable for many other uses.

134. Methods of Treatment. There are two general methods of treating wood. These are the pressure and nonpressure processes. The most effective treatment is either the full-cell or the empty-cell pressure process. In both cases the timber to be treated is stacked on steel cars and moved into a large cylinder. The cylinder is then closed and filled with a preservative; and pressure is applied to force the preservative into the wood.

The full-cell or Bethel process is used primarily when the maximum amount of retention of oil preservatives is desired or when a limited absorption of preservatives in water solution is specified. A full-cell treatment with creosote oils will provide wood with a high degree of protection. Zinc chloride, chromated zinc chloride, and sodium fluoride are usually applied by this process. After the timber has been placed in the cylinder a vacuum is applied to remove air from the cylinder and the wood. The preservative is then admitted to the cylinder without breaking the vacuum. Pressure is applied until the desired absorption is obtained. The pressure and the length of time required to complete the treatment depend upon the ease with which the wood takes the preservative. The preservative is then drawn from the cylinder and a final vacuum created to free the wood from dripping preservative.

The empty-cell process is used when it is desired to obtain a deep penetration but low net retention of preservative. No initial vacuum is used, and compressed air or air at atmospheric pressure is trapped in the wood cells when the preservative is applied under pressure. The pressure is continued until the wood refuses to take the preservative or until the desired penetration is reached. Then the entrapped air forces part of the preservative out of the wood.

There are several ways of applying preservatives without pressure, but none are as effective as the pressure processes. The most common and the one most nearly approaching the pressure processes in thoroughness of treatment is known as the hot and cold bath treatment. It consists of immersing the wood in a hot bath of preservative in an open tank and then allowing the preservative to cool. The hot bath forces some of the air and moisture out of the wood, and during the cooling period the preservative is sucked into the wood to replace the air. The length of the hot and cold bath depends upon the ease with which the wood takes treatment, but in most cases it is several hours. For well-seasoned wood that takes preservative readily a hot bath of 2 or 3 hr. and a cold bath of the same duration are usually sufficient. The temperature of the hot bath varies depending upon the type of preservative. For water solu-

tions temperatures between 170° and 200° F. are employed, and for creosote treatments the temperatures used are between 200° and 230° F. The temperature of the cold bath should be sufficient to keep creosote preservatives in a fluid condition, usually 100° to 120° F.

Other nonpressure treatments are known as the steeping and dipping processes. In the steeping process the wood is simply soaked in unheated preservative, usually mercuric chloride, for a week or more. The dipping process consists in submerging the wood in a heated preservative from 5 to 15 min. The penetration obtained in these processes is very small, usually about 1/16 in.

135. Specifications. The American Wood Preservers' Association has prepared specifications covering the details of various treating processes. These are in use by the Federal Government and have been adopted with slight revisions by the American Railway Engineering Association, the American Society for Testing Materials, and the American Society of Civil Engineers.

The A. W. P. A. specifications for No. 1 creosote oil for ties, piles, posts, structural timber, or butt treatment of poles are as follows:

"The creosote oil shall be a distillate of coal-gas tar or coke-oven tar and shall comply with the following requirements:

1. It shall contain not more than 3 per cent of water.

2. It shall contain not more than 0.5 per cent of matter insoluble in benzol.

3. The oil shall yield not more than 2 per cent coke residue.

4. The specific gravity at 38° C. compared with water at 15.5° C. shall not be less than 1.03.

5. The distillate, based on water-free oil, shall be within the following limits:

   Up to 210° C. not more than 5 per cent.
   Up to 235° C. not more than 25 per cent.

6. The tests shall be made in accordance with the standard methods of the American Wood Preservers' Association."

The A. W. P. A. specifications for zinc chloride are as follows:

"The zinc chloride shall be acid free and shall not contain more than 0.1 per cent iron. Fused or solid zinc chloride shall contain at least 94 per cent chloride of zinc. Concentrated zinc chloride solution shall contain at least 50 per cent chloride of zinc."

Table 48 is a schedule of recommended practice for the preservative treatment of timber for various uses. The Federal Government specifications and the corresponding American Wood Preservers' Association specifications are given. These specifications do not provide for treatment by nonpressure processes.

Table 48. Specifications for the Preservative Treatment of Wood

[Federal Specifications TT-W-571b]

PRESERVATIVE OILS

Form of product and service	Minimum net retention of preservative recommended: Coal-tar creosote (current issue of Fed. Spec. TT-W-556, A. W. P. A. Spec. 4)	Creosote-coal-tar solution (current issue of Fed. Spec. TT-W-566, A. W. P. A. Spec. 5e)	Creosote-petroleum solution (current issue of Fed. Spec. TT-W-568, A. W. P. A. Spec.)	Treating specification (the current issue of A. W. P. A. standard specification listed)
	Lb. per cu. ft.	*Lb. per cu. ft.*	*Lb. per cu. ft.*	
Cross ties, switch ties, bridge ties:				
Douglas fir, hemlocks, lodgepole pine	7	7	8	Use Douglas Fir Specification 38.
Oak, red, sweet gum (red gum), and black gum	7	7	8	Use Oak Specification 52.
Pine, southern yellow, ponderosa, red, jack	7	7	8	Use Southern Yellow Pine Specification 53.
Other species	7	7	8	No specification available.
Piles, for land or fresh water use:				
Douglas fir	10	10	12	41.
Oak, red and white	10	10		Use Douglas fir Specification 41.
Pine, lodgepole	10	10	12	Do.
red	12	12		Use Southern Yellow Pine Specification 39.
southern yellow	12	12		39.
Piles, marine use:				
Douglas fir	14	14		41.
Pine, southern yellow	20	20		39.
Poles:				
Cedar (butt treatment, incised)				43.
Cedar (full-length, pressure treatment with or without incision).	4			
Douglas fir	8			51.
Pine, lodgepole	8			
red	8			Use Southern Yellow Pine Specification 36.
southern yellow	8			36.
Posts, fence, round:				
Cedar	5	5	6	No standard specification.
Douglas fir	6	6	7	No post specification. Use Pole Specification 51.
Pine, all species	6	6	7	37.
Lumber and structural timber 5 in. or less in thickness (nominal dimensions) for land or fresh water use:				
Douglas fir	10	10	12	38.
Pine, southern yellow	10	10		53.
Structural timber more than 5 in. thick (nominal dimensions) for land or fresh water use:				
Douglas fir	8	8	10	38.
Pine, southern yellow	10	10		53.
Lumber and structural timber for use in salt water:				
Douglas fir	14	14		38.
Pine, southern yellow	20	20		53.

Table 48. Specifications for the Preservative Treatment of Wood (*Continued*)

PRESERVATIVE SALTS

Form of product and service	Minimum net retention of preservative recommended					Treating specification (the current issue of A W P. A Standard Specification listed)
	Zinc chloride (current issue of Fed. Spec. No. TT-W-576, and A. W. P. A. Spec 17)	Celcure (acid cupric chromate) (current issue of Fed. Spec. TT-W-546) (No A. W. P. A. Spec.)	Chromated zinc chloride (current issue of Fed. Spec. TT-W-551) (No A W. P. A Spec.)	Wolman salt Tanalith (current issue of Fed. Spec. TT-W-573) (No A. W. P. A. Spec)	Zinc meta arsenite (current issue of Fed. Spec. TT-W-581) (No A. W. P. A. Spec)	
	Lb. dry salt per cu. ft.	*Lb. dry salt per cu. ft.*	*Lb dry salt per cu. ft*	*Lb. dry salt per cu. ft.*	*Lb dry salt per cu. ft*	
Lumber and structural timber:						
Dougas fir	1	0 50	0.75	0.35	0 35	38
Pine, southern yellow	1	.50	.75	.35	.35	53

APPENDIXES

APPENDIX A

Suggested Working Stresses and Design Procedure

Wood is being substituted extensively for a great many other materials, so it in turn has become critical. Certain grades and species are under a limitation order to divert them to war uses. However, even though lumber has been curtailed for civilian use, there is still a large anticipated shortage. This shortage is not due to a reduction in our natural supply but is a direct result of the limited man power available to turn it into boards, dimension, and timbers.

A much discussed proposal for meeting the shortage is to apply an over-all increase in the working stresses for wood. Before this can be done, however, it is necessary to review the methods used in deriving the present working stresses and to investigate the true factor of safety existing in these stresses.

The present working stresses are based on the average results of tests on 250,000 small clear specimens.[1] These results have been multiplied by a number of factors to reduce them to basic stresses.[2] The first factor is based on the variation in the strength of clear wood. It is not uncommon to find one piece of wood twice as strong as another piece of the same species, although both pieces are clear, sound, and straight grained. Part of the so-called factor of safety is used up in making the working stresses safe for the weaker timbers.

In the tests at the Forest Products Laboratory on small clear specimens the maximum loads were reached in a few minutes. The duration of stress has an important bearing on the adaptation of test results to the design of different structures and members. For instance, in impact-bending, where the load is applied suddenly and then released in a few seconds, wood will resist a force more than double that required to cause failure in static bending. Conversely, wood beams under continuous loading for a period of years will fail at loads one-half to three-fourths as great as those required to produce failure in a few minutes. Chart 1 shows the relation between the modulus of rupture and duration of stress of Sitka spruce. Consequently, in deriving basic stresses, it is necessary to reduce the results obtained from short time loading tests to values applicable to long time or permanent loads. In addition, a true factor of safety must be added. As an example, in deriving the basic stresses for extreme fiber in bending, the average ultimate strength values as found from tests of clear wood in a green condition have been reduced by one-fourth to allow for the effect of variability and multiplied by nine-sixteenths to allow for long-continued stress, and then divided by one and two-thirds as a true factor of safety.

These basic stresses for clear material afford a basis for computing working stresses; they require modification only for the grade of lumber used and the conditions of exposure to which it will be subjected in service. Defects have about the same effect

[1] "Strength and Related Properties of Woods Grown in the United States," by Markwardt and Wilson, *Tech. Bull.* 479, U. S. Dept. Agr., 1935.

[2] See Table 30.

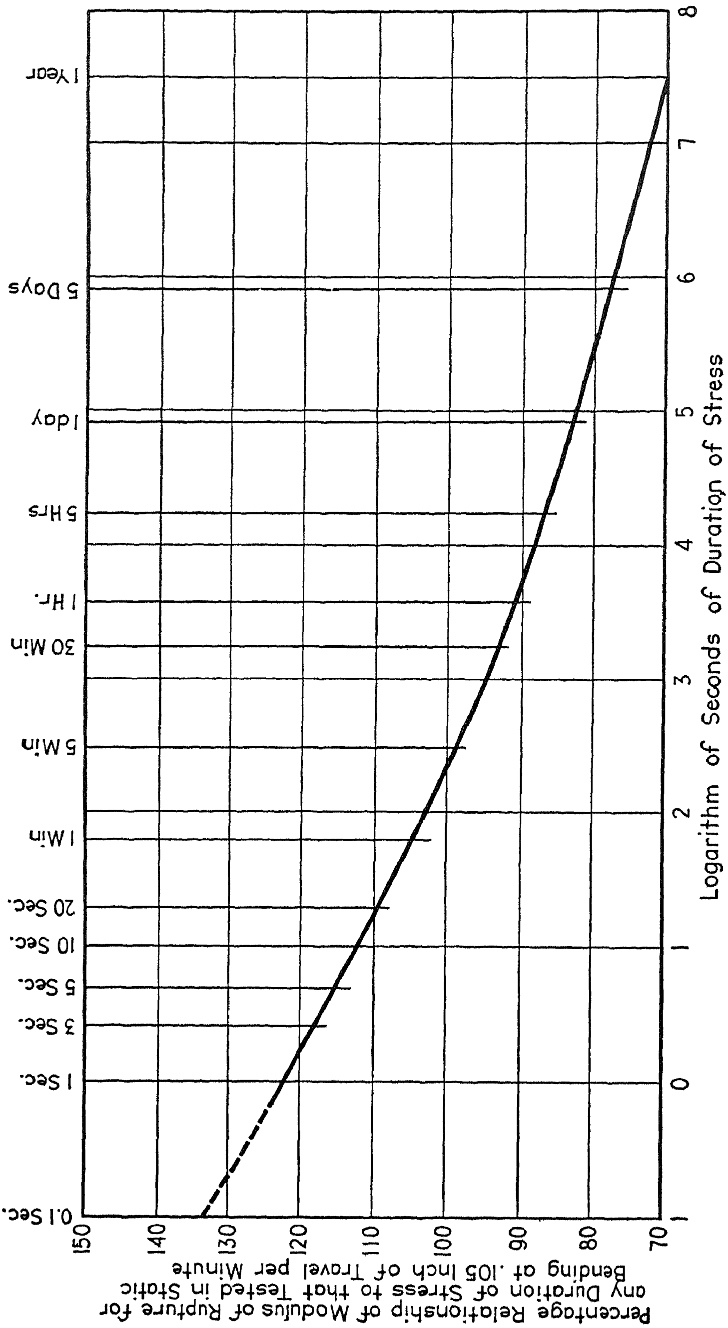


CHART 1. Variation in modulus of rupture with change in duration of stress.

on strength in all species, and methods for writing grading rules or for determining the working stress for a timber containing defects of a given size have been formulated.[1]

The working stresses for structural grades as they now exist and the use of these stresses are based on the illogical procedure of employing the same factor of safety for dead and live loads.

In an article in the October 1942 issue of "Civil Engineering" L. P. Keith has taken the values in Chart 1 and converted them from a 5-min. loading basis to a permanent loading by multiplying by the reciprocal of nine-sixteenths. The possibility of increasing working stresses for loads that will not be permanent is advocated by Mr. Keith. This can and should be done. The dead load on any structure is a fixed element and generally does not change. The present working stresses should be applied to dead loads only, and suitable increases should be made in these stresses when designing for live loads. A slight increase in these working stresses would not save nearly as much lumber as designs based on separate working stresses for dead and live loads. A suggested increase in working stresses when designing for live loads is 33⅓ per cent. In addition, more accurate methods should be employed in determining the loads acting on a structure. This is particularly true of wind loads.

The design of members on the basis of several working stresses is rather complicated and laborious. Therefore, instead of increasing the working stresses in designing for live loads, it would probably be easier to use the present working stresses and decrease the actual live load by 25 per cent.

As an example, consider the truss designed in Article 68. A comparison of the stresses in the members for full dead and live load, and for full dead and 75 per cent of the live load is given in Table 1.

Table 1. Comparison of Stresses in Members of a 50-Foot Fink Truss

Member	*Dead Load Plus Full Live Load*	*Dead Load Plus 75 Per Cent Live Load*	*Difference*
U_0	−37,030	−31,970	5,060
U_1	−35,630	−30,755	4,875
U_2	−34,150	29,450	4,700
U_3	−32,840	−28,335	4,505
L_0	34,450	29,740	4,710
L_1	29,440	25,410	4,030
L_2	19,320	16,680	2,640
V_1	−3,710	−3,200	510
V_2	−7,420	−6,410	1,010
V_3	−3,710	−3,200	510
D_1	4,970	4,290	680
D_2	4,970	4,290	680
D_3	10,130	8,740	1,390
D_4	15,060	13,000	2,060

In Article 67 the Duchemin formula is used for determining the normal wind load on the roof surface. This formula fails to take into account several factors, including (1) the suction on the leeward roof, (2) the suction on the windward side of the roof

[1] "Guide to the Grading of Structural Timbers and the Determination of Working Stresses," by T. R. C. Wilson, *Misc. Pub.* 185, U. S. Dept. Agr., 1934.

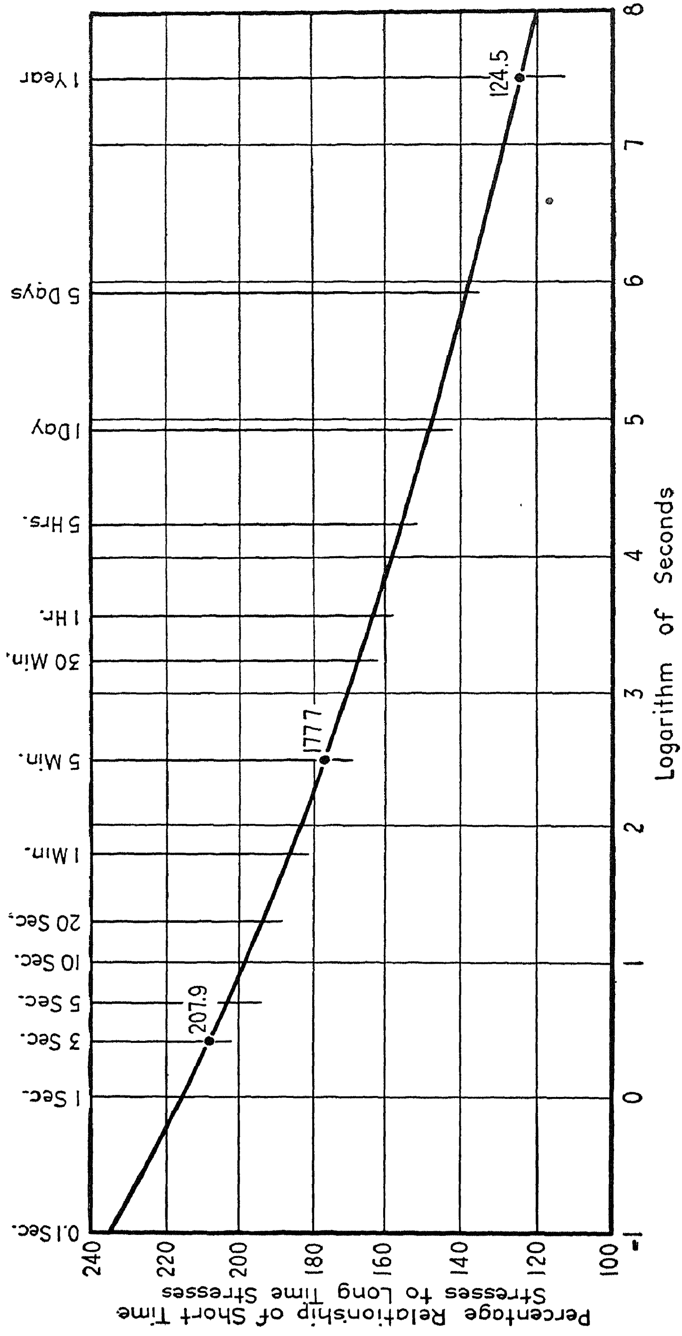


CHART 2. Variation in stresses based on duration of stress.

when the angle of inclination of the roof surface is less than 30°, and (3) internal pressure or suction. A great number of model studies have shown that suction does exist on the leeward side regardless of the slope of the roof and that for slopes less than 30° the wind acts as suction on the windward slope. Furthermore, experimental results have shown that the following relationship exists:

$$p = Cq$$

where p = wind pressure in lb. per sq. ft.,
C = coefficient depending upon the size, shape, and position of the structure in the wind,
q = velocity pressure ($0.00256V^2$) where V is the velocity of the wind in miles per hour.

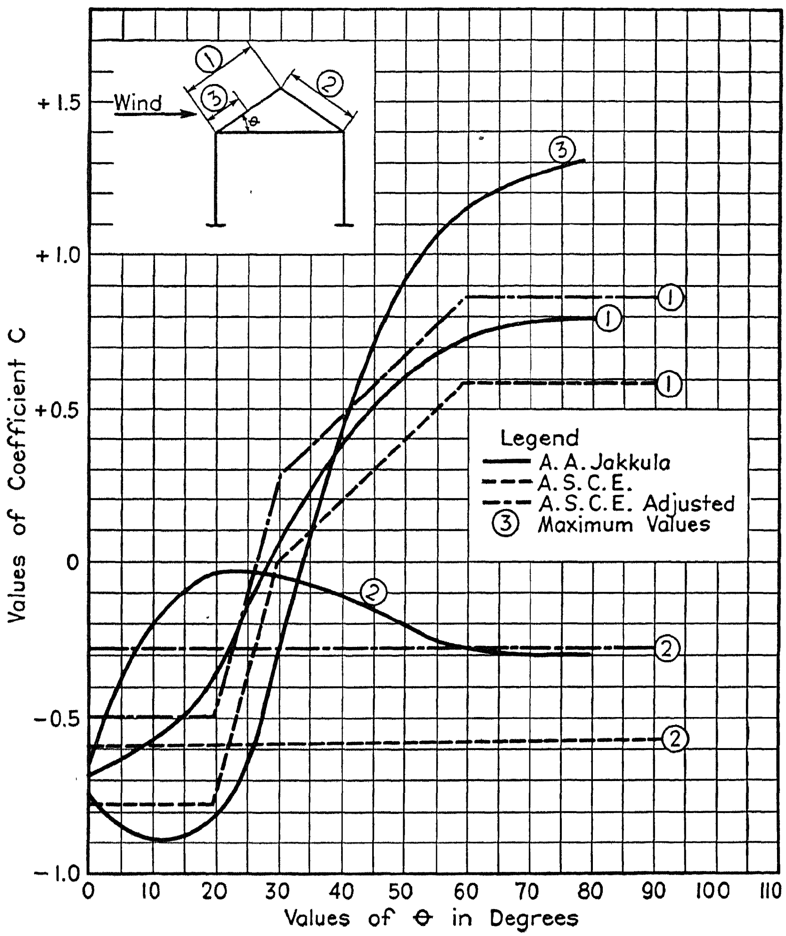


CHART 3. Values of C in the formula $p = Cq$.

In the final report [1] of Sub-Committee 31 of the Committee on Steel of the Structural Division of the American Society of Civil Engineers recommendations are made for determining the wind pressures on plane surfaces inclined to the wind. The values suggested are based on activities of the committee covering a period of 10 years in which all important experimental results of wind pressures on buildings were studied. The committee recommends a wind force of 20 lb. per sq. ft. on buildings with a vertical plane surface of 300 ft. or less. This wind force corresponds to a velocity of 77.8 mi. per hr. or a velocity pressure, q, of 15.5 lb. per sq. ft. The external pressure on the windward wall of a building is suggested as $0.8q$ and the suction on the leeward wall as $0.5q$. The average values of p, recommended for various roof slopes, may be obtained by multiplying the value of C from Chart 3 by 15.5. Two curves are shown. One represents values for an airtight building, which is normally an impossible condition, and the other represents adjusted values taking into account the internal suction.

Chart 3 also includes curves developed by Dr. A. A. Jakkula.[2] These curves include the internal suction and are based on the results of the same tests studied by Sub-Committee 31 with the exception that later tests in 1936, by Irminger and Nokkentved, are included.

Based on the recommendations of Sub-Committee 31, the following wind pressures should be used in the design of the truss in Article 68.

$$\text{Windward Slope, } P_n = -0.36 \times 15.5 = -5.58 \text{ lb. per sq. ft.}$$
$$\text{Leeward Slope, } P_n = -0.29 \times 15.5 = -4.50 \text{ lb. per sq. ft.}$$

Table 2 is a final comparison of the stresses in the members.

Table 2. Comparison of Stresses in Members of a 50-Foot Fink Truss Based on Dead Load and Various Values of Live and Wind Load

Member	Dead Load Plus Full Live Load	Dead Load Plus 75 Per Cent Live Load	Wind Load Duchemin's Formula	Wind Load A. S. C. E. Recommendations (Maximum)	Maximum Stress Col. 2 + 4	Maximum Stress Col. 3 + 5	Difference in Maximum Stresses Col. 6 − 7
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
U_0	−37,030	−31,970	−8,250	+4,910	−45,280	−27,060	18,220
U_1	−35,630	−30,755	−8,250	+4,910	−43,880	−25,845	18,035
U_2	−34,150	−29,450	−8,250	+4,910	−42,400	−24,540	17,860
U_3	−32,840	−28,335	−8,250	+4,910	−41,090	−23,425	17,665
L_0	+34,450	+29,740	+8,880	−4,540	+43,330	+25,200	18,130
L_1	+29,440	+25,410	+6,890	−3,730	+36,330	+21,680	14,650
L_2	+19,320	+16,680	+2,950	−2,120	+22,270	+14,560	7,710
V_1	− 3,710	− 3,200	−1,400	+ 600	− 5,110	− 2,600	2,510
V_2	− 7,420	− 6,410	−2,800	+1,200	−10,220	− 5,210	5,010
V_3	− 3,710	− 3,200	−1,400	+ 600	− 5,110	− 2,600	2,510
D_1	+ 4,970	+ 4,290	+1,950	− 810	+ 6,920	+ 3,480	3,440
D_2	+ 4,970	+ 4,290	+1,950	− 810	+ 6,920	+ 3,480	3,440
D_3	+10,130	+ 8,740	+3,900	−1,610	+14,030	+ 7,130	6,900
D_4	+15,060	+13,000	+5,850	−2,430	+20,910	+10,570	10,340

[1] American Society of Civil Engineers, Transactions 1940, Vol. 66N8, Part 2, pp. 1713-1739.

[2] *Structural Analysis*, by A. A. Jakkula, Edwards Brothers, Inc., 1942.

APPENDIX B

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers

Nominal Size $b \times h$ in Inches	American Standard Dressed Size (s 4 s) $b \times h$ in Inches	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
1×3	$\frac{25}{32} \times 2\frac{5}{8}$	2.05	1.18	0.90
1×4	$\frac{25}{32} \times 3\frac{5}{8}$	2.83	3.10	1.71
1×5	$\frac{25}{32} \times 4\frac{5}{8}$	3.61	6.44	2.79
1×6	$\frac{25}{32} \times 5\frac{5}{8}$	4.39	11.59	4.12
1×8	$\frac{25}{32} \times 7\frac{1}{2}$	5.86	27.47	7.32
1×10	$\frac{25}{32} \times 9\frac{1}{2}$	7.42	55.82	11.75
1×12	$\frac{25}{32} \times 11\frac{1}{2}$	8.98	99.02	17.22
$1\frac{1}{4} \times 3$	$1\frac{1}{16} \times 2\frac{5}{8}$	2.79	1.60	1.22
$1\frac{1}{4} \times 4$	$1\frac{1}{16} \times 3\frac{5}{8}$	3.85	4.22	2.33
$1\frac{1}{4} \times 5$	$1\frac{1}{16} \times 4\frac{5}{8}$	4.91	8.76	3.79
$1\frac{1}{4} \times 6$	$1\frac{1}{16} \times 5\frac{5}{8}$	5.98	15.76	5.60
$1\frac{1}{4} \times 8$	$1\frac{1}{16} \times 7\frac{1}{2}$	7.97	37.35	9.96
$1\frac{1}{4} \times 10$	$1\frac{1}{16} \times 9\frac{1}{2}$	10.09	75.91	15.98
$1\frac{1}{4} \times 12$	$1\frac{1}{16} \times 11\frac{1}{2}$	12.22	134.66	23.42
$1\frac{1}{2} \times 3$	$1\frac{5}{16} \times 2\frac{5}{8}$	3.45	1.98	1.51
$1\frac{1}{2} \times 4$	$1\frac{5}{16} \times 3\frac{5}{8}$	4.76	5.21	2.87
$1\frac{1}{2} \times 5$	$1\frac{5}{16} \times 4\frac{5}{8}$	6.07	10.82	4.68
$1\frac{1}{2} \times 6$	$1\frac{5}{16} \times 5\frac{5}{8}$	7.38	19.47	6.92
$1\frac{1}{2} \times 8$	$1\frac{5}{16} \times 7\frac{1}{2}$	9.84	46.14	12.30
$1\frac{1}{2} \times 10$	$1\frac{5}{16} \times 9\frac{1}{2}$	12.47	93.78	19.74
$1\frac{1}{2} \times 12$	$1\frac{5}{16} \times 11\frac{1}{2}$	15.09	166.35	28.93
2×2	$1\frac{5}{8} \times 1\frac{5}{8}$	2.64	0.58	0.72
2×3	$1\frac{5}{8} \times 2\frac{5}{8}$	4.27	2.45	1.87
2×4	$1\frac{5}{8} \times 3\frac{5}{8}$	5.89	6.45	3.56
2×5	$1\frac{5}{8} \times 4\frac{5}{8}$	7.52	13.40	5.79

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
2 × 6	$1\frac{5}{8} \times 5\frac{5}{8}$	9.14	24.10	8.57
2 × 7	$1\frac{5}{8} \times 6\frac{5}{8}$	10.77	39.38	11.89
2 × 8	$1\frac{5}{8} \times 7\frac{1}{2}$	12.19	57.13	15.23
2 × 9	$1\frac{5}{8} \times 8\frac{1}{2}$	13.81	83.16	19.57
2 × 10	$1\frac{5}{8} \times 9\frac{1}{2}$	15.44	116.10	24.44
2 × 12	$1\frac{5}{8} \times 11\frac{1}{2}$	18.69	205.95	35.82
2 × 14	$1\frac{5}{8} \times 13\frac{1}{2}$	21.94	333.18	49.36
2 × 16	$1\frac{5}{8} \times 15\frac{1}{2}$	25.19	504.27	65.07
2 × 18	$1\frac{5}{8} \times 17\frac{1}{2}$	28.44	725.75	82.94
2 × 20	$1\frac{5}{8} \times 19\frac{1}{2}$	31.69	1,004.10	102.98
$2\frac{1}{2} \times 4$	$2\frac{1}{8} \times 3\frac{5}{8}$	7.70	8.44	4.65
$2\frac{1}{2} \times 5$	$2\frac{1}{8} \times 4\frac{5}{8}$	9.83	17.52	7.58
$2\frac{1}{2} \times 6$	$2\frac{1}{8} \times 5\frac{5}{8}$	11.95	31.52	11.21
$2\frac{1}{2} \times 7$	$2\frac{1}{8} \times 6\frac{5}{8}$	14.08	51.49	15.54
$2\frac{1}{2} \times 8$	$2\frac{1}{8} \times 7\frac{1}{2}$	15.94	74.71	19.92
$2\frac{1}{2} \times 9$	$2\frac{1}{8} \times 8\frac{1}{2}$	18.06	108.75	25.59
$2\frac{1}{2} \times 10$	$2\frac{1}{8} \times 9\frac{1}{2}$	20.19	151.83	31.96
$2\frac{1}{2} \times 12$	$2\frac{1}{8} \times 11\frac{1}{2}$	24.44	269.32	46.84
$2\frac{1}{2} \times 14$	$2\frac{1}{8} \times 13\frac{1}{2}$	28.69	435.69	64.55
$2\frac{1}{2} \times 16$	$2\frac{1}{8} \times 15\frac{1}{2}$	32.94	659.44	85.09
$2\frac{1}{2} \times 18$	$2\frac{1}{8} \times 17\frac{1}{2}$	37.19	949.06	108.46
$2\frac{1}{2} \times 20$	$2\frac{1}{8} \times 19\frac{1}{2}$	41.44	1,313.05	134.67
3 × 1	$2\frac{5}{8} \times \frac{25}{32}$	2.05	0.10	0.27
$3 \times 1\frac{1}{4}$	$2\frac{5}{8} \times 1\frac{1}{16}$	2.79	.26	.49
$3 \times 1\frac{1}{2}$	$2\frac{5}{8} \times 1\frac{5}{16}$	3.45	.49	.75
3 × 2	$2\frac{5}{8} \times 1\frac{5}{8}$	4.27	.94	1.16
$3 \times 2\frac{1}{2}$	$2\frac{5}{8} \times 2\frac{1}{8}$	5.58	2.10	1.98
3 × 3	$2\frac{5}{8} \times 2\frac{5}{8}$	6.89	3.96	3.01
3 × 4	$2\frac{5}{8} \times 3\frac{5}{8}$	9.52	10.42	5.75
3 × 5	$2\frac{5}{8} \times 4\frac{5}{8}$	12.14	21.64	9.36

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size (s 4 s) $b \downarrow$ in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
3 × 6	$2\frac{5}{8} \times 5\frac{5}{8}$	14.77	38.93	13.84
3 × 7	$2\frac{5}{8} \times 6\frac{5}{8}$	17.39	63.61	19.20
3 × 8	$2\frac{5}{8} \times 7\frac{1}{2}$	19.69	92.29	24.61
3 × 9	$2\frac{5}{8} \times 8\frac{1}{2}$	22.31	134.34	31.61
3 × 10	$2\frac{5}{8} \times 9\frac{1}{2}$	24.94	187.55	39.48
3 × 12	$2\frac{5}{8} \times 11\frac{1}{2}$	30.19	332.69	57.86
3 × 14	$2\frac{5}{8} \times 13\frac{1}{2}$	35.44	538.21	79.73
3 × 16	$2\frac{5}{8} \times 15\frac{1}{2}$	40.69	814.60	105.11
3 × 18	$2\frac{5}{8} \times 17\frac{1}{2}$	45.94	1,172.36	133.98
3 × 20	$2\frac{5}{8} \times 19\frac{1}{2}$	51.19	1,622.00	166.36
4 × 1	$3\frac{5}{8} \times \frac{25}{32}$	2.83	0.14	0.37
4 × $1\frac{1}{4}$	$3\frac{5}{8} \times 1\frac{1}{16}$	3.85	.36	.68
4 × $1\frac{1}{2}$	$3\frac{5}{8} \times 1\frac{5}{16}$	4.76	.68	1.04
4 × 2	$3\frac{5}{8} \times 1\frac{5}{8}$	5.89	1.30	1.60
4 × $2\frac{1}{2}$	$3\frac{5}{8} \times 2\frac{1}{8}$	7.70	2.90	2.73
4 × 3	$3\frac{5}{8} \times 2\frac{5}{8}$	9.52	5.46	4.16
4 × 4	$3\frac{5}{8} \times 3\frac{5}{8}$	13.14	14.39	7.94
4 × 5	$3\frac{5}{8} \times 4\frac{5}{8}$	16.77	29.89	12.92
4 × 6	$3\frac{5}{8} \times 5\frac{5}{8}$	20.39	53.76	19.12
4 × 7	$3\frac{5}{8} \times 6\frac{5}{8}$	24.02	87.84	26.52
4 × 8	$3\frac{5}{8} \times 7\frac{1}{2}$	27.19	127.44	33.98
4 × 9	$3\frac{5}{8} \times 8\frac{1}{2}$	30.81	185.52	43.65
4 × 10	$3\frac{5}{8} \times 9\frac{1}{2}$	34.44	259.00	54.53
4 × 12	$3\frac{5}{8} \times 11\frac{1}{2}$	41.69	459.43	79.90
4 × 14	$3\frac{5}{8} \times 13\frac{1}{2}$	48.94	743.24	110.11
4 × 16	$3\frac{5}{8} \times 15\frac{1}{2}$	56.19	1,124.92	145.15
4 × 18	$3\frac{5}{8} \times 17\frac{1}{2}$	63.44	1,618.98	185.03
4 × 20	$3\frac{5}{8} \times 19\frac{1}{2}$	70.69	2,239.91	229.73
5 × 1	$4\frac{5}{8} \times \frac{25}{32}$	3.61	0.18	0.47
5 × $1\frac{1}{4}$	$4\frac{5}{8} \times 1\frac{1}{16}$	4.91	.46	.87
5 × $1\frac{1}{2}$	$4\frac{5}{8} \times 1\frac{5}{16}$	6.07	.87	1.33
5 × 2	$4\frac{5}{8} \times 1\frac{5}{8}$	7.52	1.65	2.04

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size $b \downarrow$ (s 4 s) in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
5 × $2\frac{1}{2}$	$4\frac{5}{8}$ × $2\frac{1}{8}$	9.83	3.70	3.48
5 × 3	$4\frac{5}{8}$ × $2\frac{5}{8}$	12.14	6.97	5.31
5 × 4	$4\frac{5}{8}$ × $3\frac{5}{8}$	16.77	18.36	10.13
5 × 5	$4\frac{1}{2}$ × $4\frac{1}{2}$	20.25	34.17	15.19
5 × 6	$4\frac{1}{2}$ × $5\frac{1}{2}$	24.75	62.39	22.69
5 × 7	$4\frac{1}{2}$ × $6\frac{1}{2}$	29.25	102.98	31.69
5 × 8	$4\frac{1}{2}$ × $7\frac{1}{2}$	33.75	158.20	42.19
5 × 9	$4\frac{1}{2}$ × $8\frac{1}{2}$	38.25	230.30	54.19
5 × 10	$4\frac{1}{2}$ × $9\frac{1}{2}$	42.75	321.52	67.69
5 × 12	$4\frac{1}{2}$ × $11\frac{1}{2}$	51.75	570.33	99.19
5 × 14	$4\frac{1}{2}$ × $13\frac{1}{2}$	60.75	922.64	136.69
5 × 16	$4\frac{1}{2}$ × $15\frac{1}{2}$	69.75	1,396.45	180.19
5 × 18	$4\frac{1}{2}$ × $17\frac{1}{2}$	78.75	2,009.77	229.69
5 × 20	$4\frac{1}{2}$ × $19\frac{1}{2}$	87.75	2,780.58	285.19
6 × 1	$5\frac{5}{8}$ × $\frac{25}{32}$	4.40	0.22	0.57
6 × $1\frac{1}{4}$	$5\frac{5}{8}$ × $1\frac{1}{16}$	5.98	.56	1.06
6 × $1\frac{1}{2}$	$5\frac{5}{8}$ × $1\frac{5}{16}$	7.38	1.06	1.62
6 × 2	$5\frac{5}{8}$ × $1\frac{5}{8}$	9.14	2.01	2.48
6 × $2\frac{1}{2}$	$5\frac{5}{8}$ × $2\frac{1}{8}$	11.95	4.50	4.23
6 × 3	$5\frac{5}{8}$ × $2\frac{5}{8}$	14.77	8.48	6.46
6 × 4	$5\frac{5}{8}$ × $3\frac{5}{8}$	20.39	22.33	12.32
6 × 5	$5\frac{1}{2}$ × $4\frac{1}{2}$	24.75	41.77	18.56
6 × 6	$5\frac{1}{2}$ × $5\frac{1}{2}$	30.25	76.26	27.73
6 × 7	$5\frac{1}{2}$ × $6\frac{1}{2}$	35.75	125.87	38.73
6 × 8	$5\frac{1}{2}$ × $7\frac{1}{2}$	41.25	193.36	51.56
6 × 9	$5\frac{1}{2}$ × $8\frac{1}{2}$	46.75	281.47	66.23
6 × 10	$5\frac{1}{2}$ × $9\frac{1}{2}$	52.25	392.96	82.73
6 × 12	$5\frac{1}{2}$ × $11\frac{1}{2}$	63.25	697.07	121.23
6 × 14	$5\frac{1}{2}$ × $13\frac{1}{2}$	74.25	1,127.67	167.06
6 × 16	$5\frac{1}{2}$ × $15\frac{1}{2}$	85.25	1,706.78	220.23
6 × 18	$5\frac{1}{2}$ × $17\frac{1}{2}$	96.25	2,456.38	280.73
6 × 20	$5\frac{1}{2}$ × $19\frac{1}{2}$	107.25	3,398.48	348.56
6 × 22	$5\frac{1}{2}$ × $21\frac{1}{2}$	118.25	4,555.09	423.73
6 × 24	$5\frac{1}{2}$ × $23\frac{1}{2}$	129.25	5,948.19	506.23

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size (s 4 s) $b \downarrow$ in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
7 × 2	$6\frac{5}{8} \times 1\frac{5}{8}$	10.77	2.37	2.92
7 × $2\frac{1}{2}$	$6\frac{5}{8} \times 2\frac{1}{8}$	14.08	5.30	4.99
7 × 3	$6\frac{5}{8} \times 2\frac{5}{8}$	17.39	9.99	7.61
7 × 4	$6\frac{5}{8} \times 3\frac{5}{8}$	24.02	26.30	14.51
7 × 5	$6\frac{1}{2} \times 4\frac{1}{2}$	29.25	49.36	21.94
7 × 6	$6\frac{1}{2} \times 5\frac{1}{2}$	35.75	90.12	32.77
7 × 7	$6\frac{1}{2} \times 6\frac{1}{2}$	42.25	148.76	45.77
7 × 8	$6\frac{1}{2} \times 7\frac{1}{2}$	48.75	228.52	60.94
7 × 9	$6\frac{1}{2} \times 8\frac{1}{2}$	55.25	332.65	78.27
7 × 10	$6\frac{1}{2} \times 9\frac{1}{2}$	61.75	464.41	97.77
7 × 12	$6\frac{1}{2} \times 11\frac{1}{2}$	74.75	823.81	143.27
8 × 1	$7\frac{1}{2} \times \frac{25}{32}$	5.86	0.30	0.76
8 × $1\frac{1}{4}$	$7\frac{1}{2} \times 1\frac{1}{16}$	7.97	.75	1.41
8 × $1\frac{1}{2}$	$7\frac{1}{2} \times 1\frac{5}{16}$	9.84	1.41	2.15
8 × 2	$7\frac{1}{2} \times 1\frac{5}{8}$	12.19	2.68	3.30
8 × $2\frac{1}{2}$	$7\frac{1}{2} \times 2\frac{1}{8}$	15.94	6.00	5.64
8 × 3	$7\frac{1}{2} \times 2\frac{5}{8}$	19.69	11.30	8.61
8 × 4	$7\frac{1}{2} \times 3\frac{5}{8}$	27.19	29.77	16.43
8 × 5	$7\frac{1}{2} \times 4\frac{1}{2}$	33.75	56.95	25.31
8 × 6	$7\frac{1}{2} \times 5\frac{1}{2}$	41.25	103.98	37.81
8 × 7	$7\frac{1}{2} \times 6\frac{1}{2}$	48.75	171.64	52.81
8 × 8	$7\frac{1}{2} \times 7\frac{1}{2}$	56.25	263.67	70.31
8 × 9	$7\frac{1}{2} \times 8\frac{1}{2}$	63.75	383.83	90.31
8 × 10	$7\frac{1}{2} \times 9\frac{1}{2}$	71.25	535.86	112.81
8 × 12	$7\frac{1}{2} \times 11\frac{1}{2}$	86.25	950.55	165.31
8 × 14	$7\frac{1}{2} \times 13\frac{1}{2}$	101.25	1,537.73	227.81
8 × 16	$7\frac{1}{2} \times 15\frac{1}{2}$	116.25	2,327.42	300.31
8 × 18	$7\frac{1}{2} \times 17\frac{1}{2}$	131.25	3,349.61	382.81
8 × 20	$7\frac{1}{2} \times 19\frac{1}{2}$	146.25	4,634.30	475.31
8 × 22	$7\frac{1}{2} \times 21\frac{1}{2}$	161.25	6,211.48	577.81
8 × 24	$7\frac{1}{2} \times 23\frac{1}{2}$	176.25	8,111.17	690.31

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
9 × 2	$8\frac{1}{2} \times 1\frac{5}{8}$	13.81	3.04	3.74
9 × $2\frac{1}{2}$	$8\frac{1}{2} \times 2\frac{1}{8}$	18.06	6.80	6.40
9 × 3	$8\frac{1}{2} \times 2\frac{5}{8}$	22.31	12.81	9.76
9 × 4	$8\frac{1}{2} \times 3\frac{5}{8}$	30.81	33.74	18.62
9 × 5	$8\frac{1}{2} \times 4\frac{1}{2}$	38.25	64.55	28.69
9 × 6	$8\frac{1}{2} \times 5\frac{1}{2}$	46.75	117.85	42.85
9 × 7	$8\frac{1}{2} \times 6\frac{1}{2}$	55.25	194.53	59.85
9 × 8	$8\frac{1}{2} \times 7\frac{1}{2}$	63.75	298.83	79.69
10 × 1	$9\frac{1}{2} \times \frac{25}{32}$	7.42	0.38	0.97
10 × $1\frac{1}{4}$	$9\frac{1}{2} \times 1\frac{1}{16}$	10 09	.95	1.79
10 × $1\frac{1}{2}$	$9\frac{1}{2} \times 1\frac{5}{16}$	12.47	1.79	2.73
10 × 2	$9\frac{1}{2} \times 1\frac{5}{8}$	15.44	3.40	4.18
10 × $2\frac{1}{2}$	$9\frac{1}{2} \times 2\frac{1}{8}$	20.19	7.60	7.15
10 × 3	$9\frac{1}{2} \times 2\frac{5}{8}$	24.94	14.32	10.91
10 × 4	$9\frac{1}{2} \times 3\frac{5}{8}$	34.44	37.71	20.81
10 × 5	$9\frac{1}{2} \times 4\frac{1}{2}$	42.75	72.14	32.06
10 × 6	$9\frac{1}{2} \times 5\frac{1}{2}$	52.25	131.71	47.90
10 × 7	$9\frac{1}{2} \times 6\frac{1}{2}$	61.75	217.41	66.90
10 × 8	$9\frac{1}{2} \times 7\frac{1}{2}$	71.25	333.98	89.06
10 × 10	$9\frac{1}{2} \times 9\frac{1}{2}$	90.25	678.76	142.90
10 × 12	$9\frac{1}{2} \times 11\frac{1}{2}$	109.25	1,204.03	209.40
10 × 14	$9\frac{1}{2} \times 13\frac{1}{2}$	128.25	1,947.80	288.56
10 × 16	$9\frac{1}{2} \times 15\frac{1}{2}$	147.25	2,948.07	380.40
10 × 18	$9\frac{1}{2} \times 17\frac{1}{2}$	166.25	4,242.84	484.90
10 × 20	$9\frac{1}{2} \times 19\frac{1}{2}$	185.25	5,870.11	602.06
10 × 22	$9\frac{1}{2} \times 21\frac{1}{2}$	204.25	7,867.88	731.90
10 × 24	$9\frac{1}{2} \times 23\frac{1}{2}$	223.25	10,274.15	874.40
10 × 26	$9\frac{1}{2} \times 25\frac{1}{2}$	242.25	13,126.92	1,029.56
10 × 28	$9\frac{1}{2} \times 27\frac{1}{2}$	261.25	16,464.19	1,197.40
10 × 30	$9\frac{1}{2} \times 29\frac{1}{2}$	280.25	20,323.96	1,377.90
12 × 1	$11\frac{1}{2} \times \frac{25}{32}$	8.98	0.46	1.17
12 × $1\frac{1}{4}$	$11\frac{1}{2} \times 1\frac{1}{16}$	12.22	1.15	2.16
12 × $1\frac{1}{2}$	$11\frac{1}{2} \times 1\frac{5}{16}$	15.09	2.17	3.30
12 × 2	$11\frac{1}{2} \times 1\frac{5}{8}$	18.69	4.11	5.06

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size $b \downarrow$ (s 4 s) in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
12 × $2\frac{1}{2}$	$11\frac{1}{2}$ × $2\frac{1}{8}$	24.44	9.20	8.65
12 × 3	$11\frac{1}{2}$ × $2\frac{5}{8}$	30.19	17.33	13.21
12 × 4	$11\frac{1}{2}$ × $3\frac{5}{8}$	41.69	45.65	25.19
12 × 5	$11\frac{1}{2}$ × $4\frac{1}{2}$	51.75	87.33	38.81
12 × 6	$11\frac{1}{2}$ × $5\frac{1}{2}$	63.25	159.44	57.98
12 × 7	$11\frac{1}{2}$ × $6\frac{1}{2}$	74.75	263.18	80.98
12 × 8	$11\frac{1}{2}$ × $7\frac{1}{2}$	86.25	404.30	107.81
12 × 10	$11\frac{1}{2}$ × $9\frac{1}{2}$	109.25	821.65	172.98
12 × 12	$11\frac{1}{2}$ × $11\frac{1}{2}$	132.25	1,457.51	253.48
12 × 14	$11\frac{1}{2}$ × $13\frac{1}{2}$	155.25	2,357.86	349.31
12 × 16	$11\frac{1}{2}$ × $15\frac{1}{2}$	178.25	3,568.71	460.48
12 × 18	$11\frac{1}{2}$ × $17\frac{1}{2}$	201.25	5,136.07	586.98
12 × 20	$11\frac{1}{2}$ × $19\frac{1}{2}$	224.25	7,105.92	728.81
12 × 22	$11\frac{1}{2}$ × $21\frac{1}{2}$	247.25	9,524.28	885.98
12 × 24	$11\frac{1}{2}$ × $23\frac{1}{2}$	270.25	12,437.13	1,058.48
12 × 26	$11\frac{1}{2}$ × $25\frac{1}{2}$	293.25	15,890.48	1,246.31
12 × 28	$11\frac{1}{2}$ × $27\frac{1}{2}$	316.25	19,930.34	1,449.48
12 × 30	$11\frac{1}{2}$ × $29\frac{1}{2}$	339.25	24,602.69	1,667.98
14 × 2	$13\frac{1}{2}$ × $1\frac{5}{8}$	21.94	4.83	5.94
14 × $2\frac{1}{2}$	$13\frac{1}{2}$ × $2\frac{1}{8}$	28.69	10.80	10.16
14 × 3	$13\frac{1}{2}$ × $2\frac{5}{8}$	35.44	20.35	15.50
14 × 4	$13\frac{1}{2}$ × $3\frac{5}{8}$	48.94	53.59	29.57
14 × 5	$13\frac{1}{2}$ × $4\frac{1}{2}$	60.75	102.52	45.56
14 × 6	$13\frac{1}{2}$ × $5\frac{1}{2}$	74.25	187.17	68.06
14 × 8	$13\frac{1}{2}$ × $7\frac{1}{2}$	101.25	474.61	126.56
14 × 10	$13\frac{1}{2}$ × $9\frac{1}{2}$	128.25	964.55	203.06
14 × 12	$13\frac{1}{2}$ × $11\frac{1}{2}$	155.25	1,710.98	297.56
14 × 14	$13\frac{1}{2}$ × $13\frac{1}{2}$	182.25	2,767.92	410.06
14 × 16	$13\frac{1}{2}$ × $15\frac{1}{2}$	209.25	4,189.36	540.56
14 × 18	$13\frac{1}{2}$ × $17\frac{1}{2}$	236.25	6,029.30	689.06
14 × 20	$13\frac{1}{2}$ × $19\frac{1}{2}$	263.25	8,341.73	855.56

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size $b \downarrow$ in Inches $h \downarrow$	American Standard Dressed Size $b \downarrow$ (s 4 s) in Inches $h \downarrow$	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
14 × 22	13½ × 21½	290.25	11,180.67	1,040.06
14 × 24	13½ × 23½	317.25	14,600.11	1,242.56
14 × 26	13½ × 25½	344.25	18,654.05	1,463.06
14 × 28	13½ × 27½	371.25	23,396.48	1,701.56
14 × 30	13½ × 29½	398.25	28,881.42	1,958.06
16 × 3	15½ × 2⅝	40.69	23.36	17.80
16 × 4	15½ × 3⅝	56.19	61.53	33.95
16 × 5	15½ × 4½	69.75	117.70	52.31
16 × 6	15½ × 5½	85.25	214.90	78.15
16 × 8	15½ × 7½	116.25	544.92	145.31
16 × 10	15½ × 9½	147.25	1,107.44	233.15
16 × 12	15½ × 11½	178.25	1,964.46	341.65
16 × 14	15½ × 13½	209.25	3,177.98	470.81
16 × 16	15½ × 15½	240.25	4,810.01	620.65
16 × 18	15½ × 17½	271.25	6,922.53	791.15
16 × 20	15½ × 19½	302.25	9,577.55	982.31
16 × 22	15½ × 21½	333.25	12,837.07	1,194.15
16 × 24	15½ × 23½	364.25	16,763.09	1,426.65
16 × 26	15½ × 25½	395.25	21,417.61	1,679.81
16 × 28	15½ × 27½	426.25	26,862.63	1,953.65
16 × 30	15½ × 29½	457.25	33,160.15	2,248.15
18 × 4	17½ × 3⅝	63.44	69.47	38.33
18 × 5	17½ × 4½	78.75	132.89	59.06
18 × 6	17½ × 5½	96.25	242.63	88.23
18 × 8	17½ × 7½	131.25	615.23	164.06
18 × 10	17½ × 9½	166.25	1,250.34	263.23
18 × 12	17½ × 11½	201.25	2,217.94	385.73
18 × 14	17½ × 13½	236.25	3,588.05	531.56
18 × 16	17½ × 15½	271.25	5,430.65	700.73
18 × 18	17½ × 17½	306.25	7,815.76	893.23
18 × 20	17½ × 19½	341.25	10,813.36	1,109.06

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
18 × 22	$17\frac{1}{2}$ × $21\frac{1}{2}$	376.25	14,493.46	1,348.23
18 × 24	$17\frac{1}{2}$ × $23\frac{1}{2}$	411.25	18,926.07	1,610.73
18 × 26	$17\frac{1}{2}$ × $25\frac{1}{2}$	446.25	24,181.17	1,896.56
18 × 28	$17\frac{1}{2}$ × $27\frac{1}{2}$	481.25	30,328.78	2,205.73
18 × 30	$17\frac{1}{2}$ × $29\frac{1}{2}$	516.25	37,438.88	2,538.23
20 × 4	$19\frac{1}{2}$ × $3\frac{5}{8}$	70.69	77.41	42.71
20 × 5	$19\frac{1}{2}$ × $4\frac{1}{2}$	87.75	148.08	65.81
20 × 6	$19\frac{1}{2}$ × $5\frac{1}{2}$	107.25	270.36	98.31
20 × 8	$19\frac{1}{2}$ × $7\frac{1}{2}$	146.25	685.55	182.81
20 × 10	$19\frac{1}{2}$ × $9\frac{1}{2}$	185.25	1,393.23	293.31
20 × 12	$19\frac{1}{2}$ × $11\frac{1}{2}$	224.25	2,471.42	429.81
20 × 14	$19\frac{1}{2}$ × $13\frac{1}{2}$	263.25	3,998.11	592.31
20 × 16	$19\frac{1}{2}$ × $15\frac{1}{2}$	302.25	6,051.30	780.81
20 × 18	$19\frac{1}{2}$ × $17\frac{1}{2}$	341.25	8,708.98	995.31
20 × 20	$19\frac{1}{2}$ × $19\frac{1}{2}$	380.25	12,049.17	1,235.81
20 × 22	$19\frac{1}{2}$ × $21\frac{1}{2}$	419.25	16,149.86	1,502.31
20 × 24	$19\frac{1}{2}$ × $23\frac{1}{2}$	458.25	21,089.05	1,794.81
20 × 26	$19\frac{1}{2}$ × $25\frac{1}{2}$	497.25	26,944.73	2,113.31
20 × 28	$19\frac{1}{2}$ × $27\frac{1}{2}$	536.25	33,794.92	2,457.81
20 × 30	$19\frac{1}{2}$ × $29\frac{1}{2}$	575.25	41,717.61	2,828.31
22 × 6	$21\frac{1}{2}$ × $5\frac{1}{2}$	118.25	298.09	108.40
22 × 8	$21\frac{1}{2}$ × $7\frac{1}{2}$	161.25	755.86	201.56
22 × 10	$21\frac{1}{2}$ × $9\frac{1}{2}$	204.25	1,536.13	323.40
22 × 12	$21\frac{1}{2}$ × $11\frac{1}{2}$	247.25	2,724.90	473.90
22 × 14	$21\frac{1}{2}$ × $13\frac{1}{2}$	290.25	4,408.17	653.06
22 × 16	$21\frac{1}{2}$ × $15\frac{1}{2}$	333.25	6,671.94	860.90
22 × 18	$21\frac{1}{2}$ × $17\frac{1}{2}$	376.25	9,602.21	1,097.40
22 × 20	$21\frac{1}{2}$ × $19\frac{1}{2}$	419.25	13,284.98	1,362.56
22 × 22	$21\frac{1}{2}$ × $21\frac{1}{2}$	462.25	17,806.26	1,656.40
22 × 24	$21\frac{1}{2}$ × $23\frac{1}{2}$	505.25	23,252.03	1,978.90
22 × 26	$21\frac{1}{2}$ × $25\frac{1}{2}$	548.25	29,708.30	2,330.06
22 × 28	$21\frac{1}{2}$ × $27\frac{1}{2}$	591.25	37,261.07	2,709.90
22 × 30	$21\frac{1}{2}$ × $29\frac{1}{2}$	634.25	45,996.34	3,118.40

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

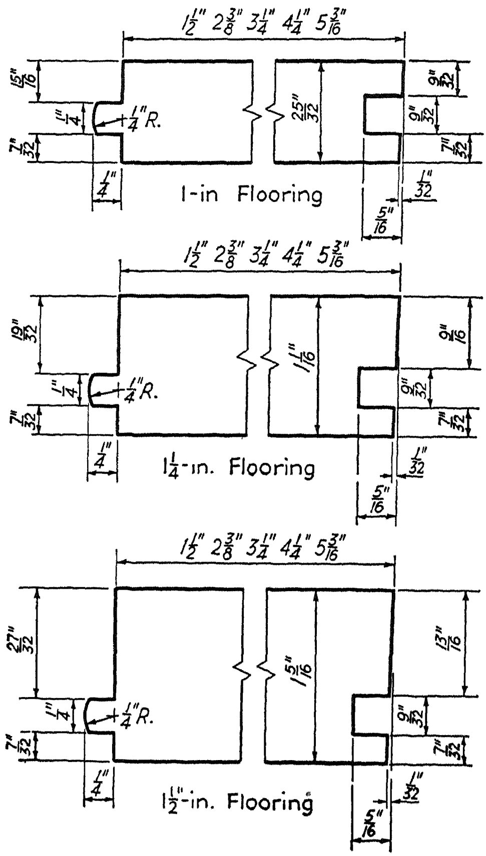
Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
24 × 6	23½ × 5½	129.25	325.82	118.48
24 × 8	23½ × 7½	176.25	826.17	220.31
24 × 10	23½ × 9½	223.25	1,679.03	353.48
24 × 12	23½ × 11½	270.25	2,978.38	517.98
24 × 14	23½ × 13½	317.25	4,818.23	713.81
24 × 16	23½ × 15½	364.25	7,292.59	940.98
24 × 18	23½ × 17½	411.25	10,495.44	1,199.48
24 × 20	23½ × 19½	458.25	14,520.80	1,489.31
24 × 22	23½ × 21½	505.25	19,462.65	1,810.48
24 × 24	23½ × 23½	552.25	25,415.01	2,162.98
24 × 26	23½ × 25½	599.25	32,471.86	2,546.81
24 × 28	23½ × 27½	646.25	40,727.21	2,961.98
24 × 30	23½ × 29½	693.25	50,275.07	3,408.48
26 × 10	25½ × 9½	242.25	1,821.92	383.56
26 × 12	25½ × 11½	293.25	3,231.86	562.06
26 × 14	25½ × 13½	344.25	5,228.30	774.56
26 × 16	25½ × 15½	395.25	7,913.23	1,021.06
26 × 18	25½ × 17½	446.25	11,388.67	1,301.56
26 × 20	25½ × 19½	497.25	15,756.61	1,616.06
26 × 22	25½ × 21½	548.25	21,119.05	1,964.56
26 × 24	25½ × 23½	599.25	27,577.98	2,347.06
26 × 26	25½ × 25½	650.25	35,235.42	2,763.56
26 × 28	25½ × 27½	701.25	44,193.36	3,214.06
26 × 30	25½ × 29½	752.25	54,553.80	3,698.56
28 × 10	27½ × 9½	261.25	1,964.82	413.65
28 × 12	27½ × 11½	316.25	3,485.34	606.15
28 × 14	27½ × 13½	371.25	5,638.36	835.31
28 × 16	27½ × 15½	426.25	8,533.88	1,101.15
28 × 18	27½ × 17½	481.25	12,281.90	1,403.65
28 × 20	27½ × 19½	536.25	16,992.42	1,742.81
28 × 22	27½ × 21½	591.25	22,775.44	2,118.65

Sizes Based on American Lumber Standards, and Properties of Boards, Dimension, and Timbers—*Continued*

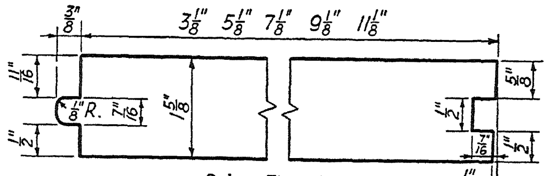
Nominal Size b ↓ in Inches h ↓	American Standard Dressed Size b ↓ (s 4 s) in Inches h ↓	Area of Section $A = bh$ Sq. In.	Moment of Inertia $I = \frac{bh^3}{12}$	Section Modulus $S = \frac{bh^2}{6}$
28 × 24	$27\frac{1}{2} \times 23\frac{1}{2}$	646.25	29,740.96	2,531.15
28 × 26	$27\frac{1}{2} \times 25\frac{1}{2}$	701.25	37,998.98	2,980.31
28 × 28	$27\frac{1}{2} \times 27\frac{1}{2}$	756.25	47,659.51	3,466.15
28 × 30	$27\frac{1}{2} \times 29\frac{1}{2}$	811.25	58,832.53	3,988.65
30 × 10	$29\frac{1}{2} \times 9\frac{1}{2}$	280.25	2,107.71	443.73
30 × 12	$29\frac{1}{2} \times 11\frac{1}{2}$	339.25	3,738.82	650.23
30 × 14	$29\frac{1}{2} \times 13\frac{1}{2}$	398.25	6,048.42	896.06
30 × 16	$29\frac{1}{2} \times 15\frac{1}{2}$	457.25	9,154.53	1,181.23
30 × 18	$29\frac{1}{2} \times 17\frac{1}{2}$	516.25	13,175.13	1,505.73
30 × 20	$29\frac{1}{2} \times 19\frac{1}{2}$	575.25	18,228.23	1,869.56
30 × 22	$29\frac{1}{2} \times 21\frac{1}{2}$	634.25	24,431.84	2,272.73
30 × 24	$29\frac{1}{2} \times 23\frac{1}{2}$	693.25	31,903.94	2,715.23
30 × 26	$29\frac{1}{2} \times 25\frac{1}{2}$	752.25	40,762.55	3,197.06
30 × 28	$29\frac{1}{2} \times 27\frac{1}{2}$	811.25	51,125.65	3,718.23
30 × 30	$29\frac{1}{2} \times 29\frac{1}{2}$	870.25	63,111.26	4,278.73

APPENDIX C

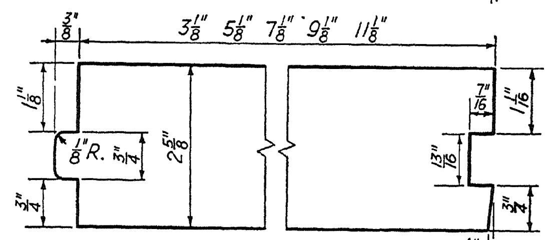
Patterns—American Lumber Standards



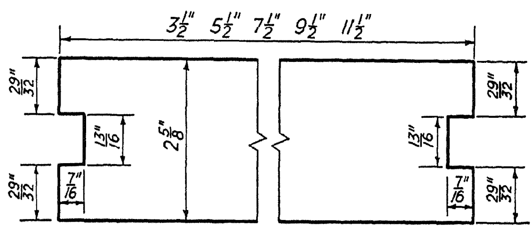
Flooring



2-in. Flooring

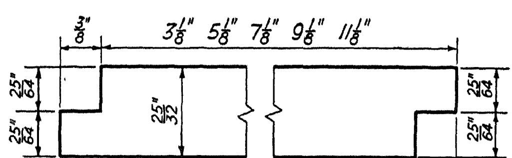


3-in. Flooring



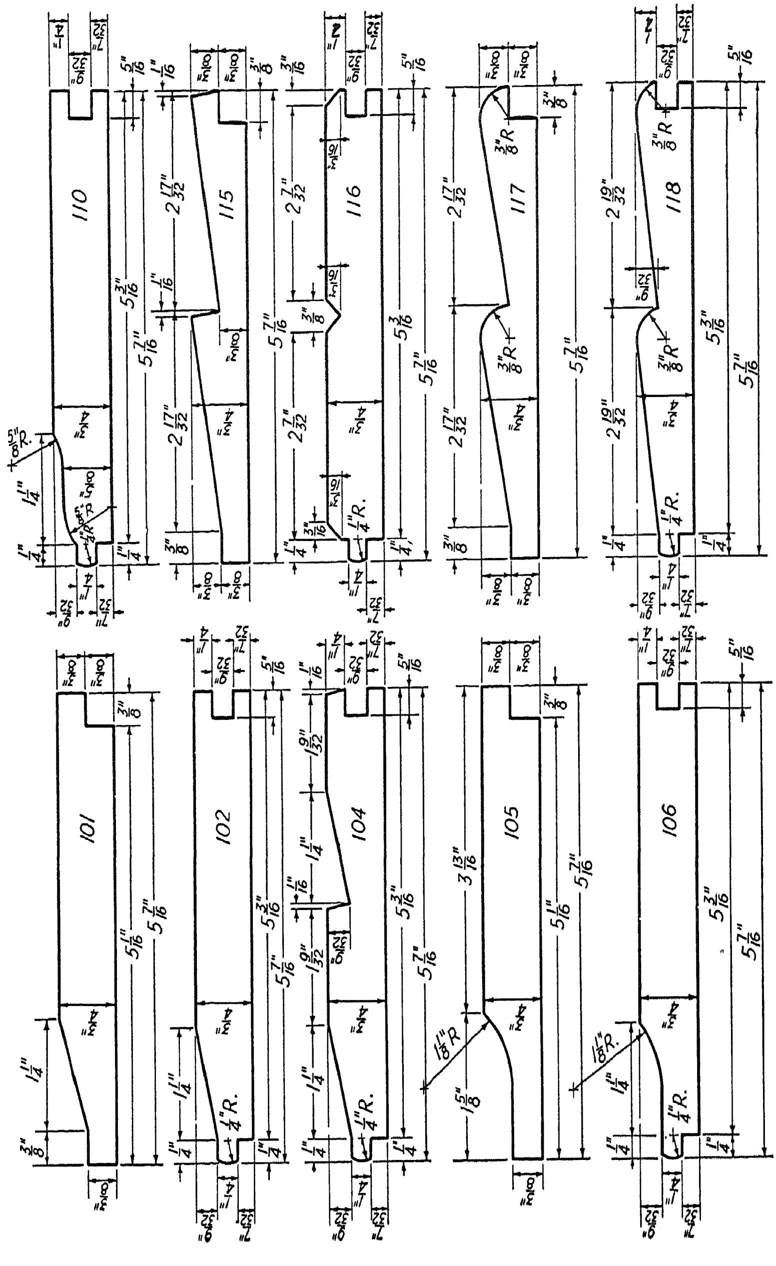
3-in. Grooved for Splines

Heavy Flooring



1-in. Shiplap

Shiplap



3/4-in. by 6-in. Drop Siding

APPENDIX D

Definitions of Terms Used in Describing Standard Grades for Lumber

Air-dried. Dried by exposure to the atmosphere without artificial heat.

American Lumber Standards. American Lumber Standards embody basic provisions for softwood lumber dealing with recognized classifications, nomenclature, basic grades, seasoning standards, sizes, uniform workings, description, measurement, tally, shipping provisions, grade marking, tally cards, and inspection of lumber. The primary purpose of these standards is to serve as a guide or basic examples in the preparation or revision of the grading rules of the various lumber manufacturers' associations; their use as a framework for such rules will eliminate differences often existing.

Annual growth ring. *See* Ring, annual growth.

Bastard sawn. Hardwood lumber in which the annual rings make angles of 30° to 60° with the surface of the piece.

Beams and stringers. Lumber of rectangular cross section, 5 or more in. thick and 8 or more in. wide, graded with respect to its strength when loaded on the narrow face.

Bird's-eye. A small central spot with the wood fibers arranged around it in the form of an ellipse so as to give the appearance of an eye.

Boards. *See* Lumber.

Bow. A deviation flatwise from a straight line drawn from end to end of a piece. It is measured at the point of greatest distance from the straight line.

Boxed pith. When the pith is between the four faces on an end of a piece.

Brashness. A condition of wood characterized by low resistance to shock and by abrupt failure across the grain without normal splintering.

Broad-leaved trees. *See* Hardwoods.

Burl. A large wartlike excrescence on a tree trunk. It contains the dark piths of a large number of buds, which rarely develop.

Cambium. The layer of tissue just beneath the bark from which the new wood and bark cells of each year's growth develop.

Cell. A general term for the minute units of wood structure. It includes fibers, vessels, and other elements of diverse structure and functions.

Cellulose. The carbohydrate that is the principal constituent of wood and forms the framework of the cells.

Check. A lengthwise separation of the wood, which usually occurs across the rings of annual growth.

Close grain. *See* Grain.

Coarse grain. *See* Grain.

Conifer. *See* Softwood.

Connectors. *See* Timber Connectors.

Crook. A deviation edgewise from a straight line drawn from end to end of a piece. It is measured at the point of greatest distance from the straight line.

Cross break. A separation of the wood cells across the grain, such as may be due to tension resulting from unequal shrinkage or mechanical stress.

Cross grain. *See* Grain.

Cup. A curve in a piece across the grain or width of a piece. It is measured at the point of greatest deviation from a straight line drawn from edge to edge of a piece.

Decay. Disintegration of wood substance due to the action of wood-destroying fungi.

Incipient decay. The early stage of decay in which the disintegration has not proceeded far enough to soften or otherwise impair the hardness of the wood perceptibly.

Typical or advanced decay. The stage of decay in which the disintegration is readily recognized because the wood has become punky, soft, spongy, stringy, pitted, or crumbly.

Density. The mass of a body per unit volume. It is commonly although erroneously considered synonymous with specific gravity.

Density rule. Rules for estimating the density of wood based on percentage of summerwood and rate of growth.

Diagonal grain. *See* Grain.

Diffuse-porous woods. Hardwoods in which the pores are practically uniform in size throughout each annual ring, or decrease slightly in size toward the outer border of the ring.

Dimension. *See* Lumber.

Dimension stock. Squares or flat stock usually in pieces under the minimum sizes admitted in standard lumber grades, rough, dressed, green or dry, cut to the approximate dimension required for the various products of woodworking factories.

Dote. "Dote," "doze," and "rot" are synonymous with "decay."

Dry rot. A term loosely applied to many types of decay but especially to that which, when in an advanced stage, permits the wood to be easily crushed to a dry powder. The term is actually a misnomer for any decay, since all fungi require considerable moisture for growth, and the wood must have been moist at the time the "dry rot" occurred.

Durability. A general term for permanence or lastigness. Frequently used to refer to the degree of resistance of a species or of an individual piece of wood to decay. In this connection "resistance to decay" is a more specific term.

Edge grain. *See* Grain.

Empty-cell process. Any process for impregnating wood with preservatives or chemicals in which air is imprisoned in the wood under the pressure of the entering preservative and then expands, when the pressure is released, and drives out part of the injected preservative.

Encased knot. *See* Knot.

Extractives. Substances in wood, not an integral part of the cellular structure, that can be dissolved out with inert solvents.

Equilibrium moisture content. The moisture content at which wood neither gains nor loses moisture when surrounded by air at a given relative humidity and temperature.

Face width. The width of the face of a piece of dressed and matched or shiplapped lumber, not including the width of the tongue or lap. The amount of flooring, ceiling, siding, or other matched items required to cover a given area should be computed on the basis of the face width of the pieces. *See* Over-all width.

Factory and shop lumber. *See* Lumber.

Fiber. A wood fiber is a comparatively long (1/25 or less to 1/8 in.), narrow, tapering cell closed at both ends.

Fiber-saturation point. The stage in the drying or in the wetting of wood at which the cell walls are saturated and the cell cavities are free from water.

Figure. The pattern produced in a wood surface by irregular coloration and by annual growth rings, rays, knots, and such deviations from regular grain as interlocked and wavy grain.

Fireproofing. Making wood resistant to fire to a degree that makes it difficult to ignite and keeps it from supporting its own combustion.

Flat grain. *See* Grain, Plain sawn.

Flitch. A thick piece of lumber with wane (bark) on one or more edges.

Full-cell process. Any process for impregnating wood with preservatives or chemicals in which a vacuum is drawn to remove air from the wood before admitting the preservative.

Grade. Any of the quality classes into which products are segregated.

Grain. The direction, size, arrangement, appearance, or quality of the fibers in wood.

Close-grained wood. Wood with narrow and inconspicuous annual rings. The term is sometimes used to designate wood having small and closely spaced pores, but in this sense the term "fine-textured" is more often used.

Coarse-grained wood. Wood with wide and conspicuous annual rings in which there is considerable difference between springwood and summerwood. The term is also used to designate wood with large pores, but in this sense the term "coarse-textured" is more often used.

Cross grain. Wood in which the cells or fibers do not run parallel with the axis or sides of a piece.

Diagonal grain. Annual rings at an angle with the axis of a piece as a result of sawing at an angle with the bark of the tree.

Edge grain (vertical grain). Lumber in which the rings (so-called grain) form an angle of 45° or more with the surface of the piece.

Flat grain. Lumber in which the rings form an angle of less than 45° with the surface of the piece.

Heartwood. The inner core of the tree trunk comprising the annual rings containing non-living elements; usually darker in color than sapwood.

Interlocked grained wood. Wood in which the fibers are inclined in one direction in a number of rings of annual growth, then gradually reverse and are inclined in an opposite direction in succeeding growth rings, then reverse again.

Open-grained wood. Common classification of painters for wood with large pores, also known as "coarse-textured."

Plain sawn. Another term for flat grain, and used generally in hardwoods.

Quarter sawn. Another term for edge grain, and used generally in hardwoods.

Spiral grain. A type of growth in which the fibers take a spiral course about the bole of a tree instead of the normal vertical course. The spiral may extend right handed or left handed around the tree trunk.

Vertical grain. Another term for edge grain.

Wavy-grained wood. Wood in which the fibers collectively take the form of waves or undulations.

Green. Unseasoned, wet.

Growth ring. *See* Ring, annual growth.

Hardwoods. The botanical group of trees that are broadleaved. The term has no reference to the actual hardness of the wood.

Heart, heartwood. The wood, the cells of which no longer participate in the life processes of the tree, extending from the pith to the sapwood. Heartwood is usually darker in color than sapwood.

Imperfect manufacture. Includes all defects or blemishes which are pro-

duced in manufacturing, such as chipped grain, loosened grain, raised grain, torn grain, skips in dressing, hit or miss, variation in sawing, miscut lumber, machine burn, machine gouge, mismatching, and insufficient tongue or groove.

Interlocking grain. *See* Grain.

Joists and planks. Lumber of rectangular cross section, 2 in. to but not including 5 in. thick and 4 or more in. wide, graded with respect to its strength in bending when loaded either on the narrow face as joist or on the wide face as plank.

Kiln dried. *See* Seasoning.

Knot. A branch or limb, embedded in the tree, which has been cut through in the process of lumber manufacture. Knots are classified according to size, form, quality, and occurrence. To determine the size of a knot average maximum length and maximum width unless otherwise specified.

Encased knot. One whose rings of annual growth are not intergrown and homogeneous with those of the surrounding wood. The encasement may be partial or complete; of pitch or bark.

Intergrown knot. A knot whose rings of annual growth are completely intergrown with those of the surrounding wood.

Laminated wood. An assembly of wood built up of plies or laminations that have been joined either with glue or with mechanical fastenings. The term is most frequently applied where the plies are too thick to be classified as veneer and where the grain of all plies is parallel.

Lignin. A principal constituent of wood, second in quantity to cellulose. It incrusts the cell walls and cements the cells together.

Lumber. Lumber is the product of the saw and planing mill not further manufactured than by sawing, resawing, and passing lengthwise through a standard planing mill, cross-cutting to length, and working. Lumber of thickness not in excess of $\frac{1}{4}$ in. to be used for veneering is classified as veneer.

Factory and shop lumber. Lumber intended to be cut up for use in further manufacture. It is graded on the basis of the percentage of the area which will produce a limited number of cuttings of a specified, or a given minimum, size and quality.

Yard lumber. Lumber of all sizes and patterns which is intended for general building purposes. The grading of yard lumber is based on the intended use of the particular grade and is applied to each piece with reference to its size and length when graded without consideration to further manufacture.

STRIPS. Yard lumber less than 2 in. thick and less than 8 in. wide.

BOARDS. Yard lumber less than 2 in. thick and 8 or more in. wide.

DIMENSION. All yard lumber except boards, strips and timbers; that is yard lumber from 2 in. to but not including 5 in. thick, and of any width.

TIMBERS. Lumber 5 or more in. in least dimension.

Millwork. Generally all building materials made of finished wood and manufactured in millwork plants and planing mills are included under the term millwork, i.e., doors, window and door frames, sash, blinds, porch work, mantels, panel work, stairways, and special woodwork. It does not include finish dressed four sides, or siding, or partition, which are items of yard lumber.

Moisture content of wood. Weight of the water contained in the wood usually expressed in percentage of the weight of the oven-dry wood.

Moisture gradient. A condition of graduated moisture content between the inner and outer portions of a material, such as wood, due to the losing or absorbing of moisture.

Moisture-proofing. Making wood resistant to change in moisture content, especially to entrance of moisture.

Open-grain. *See* Grain.

Over-all width. The total of a wide piece of dressed and matched or shiplapped lumber including the width of the tongue or lap. The amount of such lumber required to cover a given area should not be computed on the basis of the over-all width since the tongue or lap is the means of joining the pieces and does not "cover" any surface. *See* face width.

Peck (found in cedar and cypress). Channeled or pitted areas or pockets of localized decay.

Pitch pocket. A well-defined opening between rings of annual growth, usually containing, or which has contained, more or less pitch, either solid or liquid. Bark also may be present in the pocket.

Pith. The small soft core occurring in the structural center of a log. The wood immediately surrounding the pith often contains small checks, shakes, or numerous pin knots, and is discolored; any such combination of characteristics is known as heart center.

Plywood. A piece of wood made of three or more layers of veneer joined with glue, and usually laid with the grain of adjoining plies at right angles. Almost always an odd number of plies are used to secure balanced construction.

Plain sawn. *See* Grain.

Planing mill products. Products worked to pattern, such as flooring, ceiling, and siding.

Pocket rot. Typical decay which appears in the form of a hole, pocket, or area of soft rot usually surrounded by apparently sound wood.

Pore. *See* Vessel.

Posts and timbers. Lumber of square or approximately square cross section, 5 by 5 in. and larger, graded primarily for use as posts or columns carrying longitudinal load but adapted for miscellaneous uses in which strength in bending is not especially important.

Preservative. With reference to wood, any substance applied to or injected into wood to protect it from attack of fungi, insects, or marine animals.

Quarter sawn. *See* Grain.

Radial. Coincident with a radius from the axis of the tree or log to the circumference.

Rate of growth. The rate at which a tree has laid on wood, measured radially in the trunk or in lumber cut from the trunk. The unit of measure in use is the number of annual growth rings per inch.

Rays, wood. Strips of cells extending radially within a tree and varying in height from a few cells in some species to 4 in. or more in oak. The rays serve primarily to store food and transport it horizontally in the tree.

Ring, annual growth. The growth layer put on in a single growth year.

Ring-porous woods. A group of hardwoods in which the pores are comparatively large at the beginning of each annual ring and decrease in size more or less abruptly toward the outer portion of the ring, thus forming a distinct inner zone of pores known as the springwood and the outer zone with smaller pores known as the summerwood.

Rot. *See* Decay.

Sap. All the fluids in a tree, except special secretions and excretions, such as oleoresin.

Sapwood. The outer layers of growth in a tree, exclusive of bark, which contains living elements; usually lighter in color than heartwood. Under most conditions sapwood is more susceptible to decay than heartwood; as a rule, it is more permeable to liquids than heartwood. Sapwood is not essentially weaker or stronger than heartwood of the same species.

Seasoning. The evaporation or extraction of moisture from green or partly dried wood.

Air dried or air seasoned. Dried by exposure to the atmosphere usually in a yard, without artificial heat.

Kiln-dried. Dried in a kiln with the use of artificial heat.

Second growth. Timber that has grown after the removal by any means of all or a large portion of the previous stand.

Shake. A separation along the grain, most of which occurs between the rings of annual growth.

Shop lumber. *See* Lumber.

Side-cut (pithless). The term used when the pith is not present in a piece.

Softwood. One of the group of trees which has needlelike or scalelike leaves, often referred to as conifers. The term "softwood" has no reference to the softness of the wood.

Specific gravity. The ratio of the weight of a body to the weight of an equal volume of water at some standard temperature.

Spiral grain. *See* Grain.

Split (through check). A lengthwise separation of the wood which occurs usually across the rings of annual growth, extending from one surface through the piece to the opposite surface, or to an adjoining surface.

Springwood. The more or less open and porous tissue marking the inner part of each annual ring, formed early in the period of growth. It is usually less dense and weaker mechanically than summerwood.

Stain, blue. A bluish or grayish discoloration of the sapwood caused by the growth of certain moldlike fungi on the surface and in the interior of the piece.

Strength. The properties of wood which enable it to resist different forces or loads. Strength may apply to any one of the mechanical properties, such as strength in bending, hardness, and strength in compression.

Strips. *See* Lumber.

Structural lumber. Lumber that is 2 or more in. thick and 4 or more in. wide, intended for use where working stresses are required. The grading of structural lumber is based on the strength of the piece and the use of the entire piece.

Summerwood. The dense, fibrous outer portion of each annual ring, usually without conspicuous pores, formed late in the growing period, not necessarily in summer. It is usually more dense and stronger mechanically than springwood.

Tangential. Strictly, coincident with a tangent at the circumference of a tree or log, or parallel to such a tangent. In practice, however, it often means roughly coincident with a growth ring.

Texture. A term often used interchangeably with grain.

Timber. A broad term including standing trees, and certain products thereof.

Round timber. Timber used in the original round form, such as poles, piling, and mine timbers.

Standing timber. Timber still on the stump.

Timber connectors. Rings and dowels of metal or wood in adjoining mem-

bers, placed in pre-cut grooves or holes in timber framing, and the timber members then drawn and held tightly together by bolts. These devices provide an efficient means of transferring load at the joints of structural members.

Timbers. Lumber 5 in. or larger in least dimension.

Twist. A form of warp resulting in distortion caused by the turning or winding of the edges of a board.

Veneer. Thin sheets of wood.

Vertical grain. *See* Grain.

Vessels. Wood cells of comparatively large diameter which have open ends and are set one above the other, forming continuous tubes. The openings of the vessels on the surface of a piece of wood are usually referred to as pores.

Virgin growth. The original growth of mature trees.

Wane. Bark, or lack of wood or bark, from any cause, on edge or corner of a piece.

Warp. Any variation from a true or plane surface. It includes bow, crook, cup, or any combination thereof.

Wavy grain. *See* Grain.

Weathering. The mechanical and chemical disintegration and discoloration of the surface of wood that is caused by exposure to light, and by the alternate shrinking and swelling of the surface fibers with continual changes in moisture content due to weather changes. Weathering does not include decay.

Workability. The degree of ease and smoothness of cut obtainable with hand or machine tools.

Working of wood. Change in the dimensions of a piece of wood with change in moisture content.

Yard lumber. *See* Lumber.

APPENDIX E

Standard Lumber Abbreviations

AD. = Air dried.
a.l. = All lengths.
av. = Average.
av.w. = Average width.
av.l. = Average length.
a.w. = All widths.
B1S = Beaded one side.
B2S = Beaded two sides.
bd. = Board.
bd.ft. = Board foot; i.e., an area of 1 sq. ft. by 1 in. thick, or the product thereof.
bdl. = Bundle.
Bev. = Beveled.
b.m. = Board (foot) measure.
Btr. = Better.
Clg. = Ceiling.
Clr. = Clear.
CM. = Center matched; i.e., the tongue and groove joints are worked along the center of the edges of the piece.
Com. = Common.
Cag. = Casing.
cu.ft. = Cubic foot.
Cust. = Custom (sawed).
D&CM. *See* S&CM.
D&H = Dressed and headed; i.e., dressed one or two sides and worked to tongue and groove joints on both the edge and the ends.
D&M. = Dressed and matched; i.e., dressed one or two sides and tongued and grooved on the edges. The match may be center or standard.
D&SM. *See* S&SM.
D2S&CM. *See* S2S&CM.
D2S&M. *See* S2S&M.
D2S&SM. *See* S2S&SM.
Dim. = Dimension.
D.S. = Drop siding.
E. = Edge.
E&CB1S = Edge and center bead one side; surfaced one or two sides and with a longitudinal edge and center bead on a surfaced face.
E&CB2S = Edge and center bead two sides; all four sides surfaced and with a longitudinal edge and center bead on the two faces.
ECM = Ends center matched.
E&CV1S = Edge and center V one side; surfaced one or two sides and with a longitudinal edge and center V-shaped groove on a surfaced face.
E&CV2S = Edge and center V two sides; all four sides surfaced and with a longitudinal edge and center V-shaped groove on each of the two faces.
E.G. = Edge grain.
EM. = End matched—either center or standard.
ESM. = Ends standard matched.
FAS. = Firsts and seconds—a combined grade of the two upper grades of hardwoods.
f.bk. = Flat back.
fcty. = Factory (lumber).
F.G. = Flat grain.
Flg. = Flooring.

Frm. = Framing.
ft. = Foot or feet. Also one accent ('). *See* Symbols.
ft.b.m. = Feet board measure.
ft.s.m. = Feet surface measure.
G.R. = Grooved roofing.
H.bk. = Hollow back.
Hdwd. = Hardwood.
Hrt. = Heart.
Hrtwd. = Heartwood.
1s&2s. = Ones and twos—a combined grade of the hardwood grades of firsts and seconds.
in. = Inch or inches. Also two accent marks ("). *See* Symbols.
KD. = Kiln-dried.
k.d. = Knocked down.
lbr. = Lumber.
l.c.l. = Less carload lots.
lgth. = Length.
lgr. = Longer.
lin.ft. = Linear foot; i.e., 12 inches.
Lng. = Lining.
LR = Log run.
LR,MCO. = Log run, mill culls out.
Lth. = Lath.
M. = Thousand.
M.b.m. = Thousand (feet) board measure.
MCO. = Mill culls out.
Merch. = Merchantable.
m.l. = Mixed lengths.
Mldg. = Moulding.
MR. = Mill run.
M.s.m. = Thousand (feet) surface measure.
m.w. = Mixed widths.
No. = Number.
Ord. = Order.
P. = Planed.
Pat. = Pattern.
Pln. = Plain, as plain sawed.
Pn. = Partition.
Prod. = Production.
Qtd. = Quartered—when referring to hardwoods.
rdm. = Random.
res. = Resawed.
Rfg. = Roofing.
Rfrs. = Roofers.
rip. = Ripped.
r.l. = Random lengths.
rnd. = Round.
R.Sdg. = Rustic siding.
r.w. = Random widths.
S1E. = Surfaced one edge.
S2E. = Surfaced two edges.
S1S. = Surfaced one side.
S2S. = Surfaced two sides.
S1S1E. = Surfaced one side and one edge.
S2S1E. = Surfaced two sides and one edge.
S1S2E. = Surfaced one side and two edges.
S4S. = Surfaced four sides.
S4SCS. = Surfaced four sides with a calking seam on each edge.
S&CM. = Surfaced one or two sides and center matched.
S&M. = Surfaced and matched; i.e., surfaced one or two sides and tongued and grooved on the edges. The match may be center or standard.
S&SM. = Surfaced one or two sides and standard matched.
S2S&CM. = Surfaced two sides and center matched.
S2S&M. = Surfaced two sides and (center or standard) matched.
S2S&SM. = Surfaced two sides and standard matched.
Sap. = Sapwood.
SB. = Standard bead.
Sd. = Seasoned.
Sdg. = Siding.
Sel. = Select.
s.f. = Surface foot; i.e., an area of one square foot.
Sftwd. = Softwood.
Sh.D. = Shipping dry.
Ship. = Shipment or shipments.
Shlp. = Shiplap.
s.m. = Surface measure.

SM. = Standard matched.
snd. = Sound.
sq. = Square.
Sq.E. = Square edge.
sqrs. = Squares.
Std. = Standard.
stnd. = Stained.
stk. = Stock.
stp. = Stepping.
Symbols: ″ = inch or inches, as 12″.
′ = foot or feet, as 12′.
x = by, as a 6x8 timber.
When referring to the thickness of lumber, 4/4, 5/4, 6/4, 8/4, etc. = 1 inch, 1¼ inches, 1½ inches, 2 inches, etc.
T&G = Tongued and grooved.
TB&S. = Top, bottom, and sides.
Tbrs. = Timbers.
V1S. = V one side, i.e., a longitudinal V-shaped groove on one face of a piece of lumber.
V2S. = V two sides; i.e., a longitudinal V-shaped groove on two faces of a piece of lumber.
V.G. = Vertical grain.
w.a.l. = Wider, all lengths.
Wth. = Width.
wdr. = Wider.
wt. = Weight.

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